REPORT

TO

EASTWOOD CENTRE DEVELOPMENTS PTY LTD

ON

GEOTECHNICAL INVESTIGATION

FOR

PROPOSED REDEVELOPMENT OF SHOPPING CENTRE

AT

CORNER RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

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EXECUTIVE SUMMARY

Geotechnical investigations have been carried out in two stages at the proposed development site. Two, approximately 20m deep boreholes were drilled in the south-west portion of the site in April 2004. A further five boreholes, about 10m or 14m deep, have now been drilled to provide a better coverage of the site. The boreholes have revealed a subsurface profile generally consisting of pavements, shallow, and in places, deep fill over residual silty clays and clays, which grade into weathered shale/siltstone bedrock at depths between about 5m and 7m below existing levels. Subsequent monitoring indicated shallow or elevated groundwater levels in the natural silty clays and clays, and groundwater flowing, presumably from defects within the shale/siltstone bedrock.

The proposed development will presumably involve substantial changes to the site including demolition of the existing structures and pavements, and excavations of large volumes of soil and rock. Good engineering design, construction and maintenance practices should be adopted to maintain stability to adjoining buildings and structures during excavation and in the long term, as well as reducing the risk of vibrational damage to adjoining buildings and structures during excavation. Insufficient space is available to form temporary batter slopes around most sides of the excavation for the presently proposed layout of the basement, requiring the use of a suitable retention system installed prior to commencing bulk excavation.

The excavations are likely to encounter groundwater seepage requiring some dewatering during construction and the provision of drainage behind the permanent basement walls and below its floor.

The proposed building should be founded on the underlying bedrock.

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1 INTRODUCTION

This report presents the results of a geotechnical investigation for the proposed re-development of the shopping centre at the corner of Rutledge and Trelawney Streets, Eastwood, NSW. The investigation was commissioned by Mr Bradley Chan of Eastwood Centre Developments Pty Ltd on the basis of our proposal, Ref: P14633V and the Short Form Consultancy Agreement. This report amplifies the preliminary information forwarded on 19 October 2007.

The supplied architectural design brief indicates that it is proposed to construct a multi-purpose retail and residential building with driveway access from Trelawney Street. The building will comprise two retail and four parking levels, with two levels below Rowe Street and three levels below Rutledge Street. There will be up to four separate residential towers over the podium ranging from two to six storeys. The basement floor is to have a finished floor level at RL 62.55m. The construction of the new building will involve the demolition of the several buildings, including the Eastwood Shopping Centre, and other structures and improvements on the site. Bulk excavation will be required to form the proposed basement, grading from about 6m to approximately 12m (maximum) depth. Presumably, locally deeper excavation would be required for the lift wells, service trenches and for footings. Column loads of the order of 3,000kN to 13,000kN have been advised.

Douglas Partners Pty Ltd (DP) have previously undertaken a preliminary geotechnical investigation at the western end of the proposed development site; details are given in their report (Project 36766, issued in April 2004). A copy of the borehole logs is contained in the Appendix to this report. Their investigation comprised two boreholes to depths between 19.7m and 20.0m below ground surface at that time. The subsurface profile generally consisted of pavements, fill, clays and shaly clays which graded into siltstone at approximate 6.7m depth. Further details are discussed in Section 3.2 of this report.

The scope of the investigation was to obtain geotechnical information on subsurface conditions at five additional locations as a basis for comments and recommendations on earthworks and subgrade preparation, excavation conditions, groundwater, excavation retention, footings, basement floor slabs, pavements, soil aggressivity and on any further geotechnical work deemed necessary during construction.

2 INVESTIGATION PROCEDURE

The fieldwork for the current investigation was carried out using our JK250 crawler drill rig or a Melvelle portable rig, where there was restricted access due to the present site development. Prior to drilling, the borehole locations were electromagnetically scanned for buried services and all concrete surfacing was cored using a diatube and water flush.

The three boreholes (BH101, BH104 and BH105) were drilled using spiral augers, to depths between 6.66m to 7.55m below existing levels. These boreholes were then extended further into the bedrock by rotary diamond coring techniques, using an NMLC triple tube core barrel with water flush, to termination at depths between 14.13m and 14.30m.

BHs 102 and 103 were drilled initially using a hand auger to refusal, then by rotary wash boring down to the shale at 7.15m and 6.70m, respectively, after which the rock was cored to termination at approximately 10m below existing levels. Prior to drilling, a Dynamic Cone Penetration test was carried out at BHs 102 and 103 to refusal at 2.69m and 1.96m depth.

The boreholes were drilled at the locations as shown on the attached Figure 1 and were set out using taped measurements from existing surface features and apparent site boundaries. The surface RLs shown on the borehole logs were estimated by interpolation between spot levels indicated on the provided survey information shown on Clement & Reid Pty Ltd Drawing No. 17677-1 dated 3 July 2006. The survey datum is the Australian Height Datum (AHD).

The composition of the subsurface materials was established by logging the materials recovered from the boreholes. The relative density/strength of the subsoils was assessed from the Standard Penetration (SPT) 'N' values, augmented by hand penetrometer readings on the recovered split tube cohesive soil samples. The strength of the underlying bedrock which was auger drilled was assessed by observation of the drilling resistance when using a tungsten carbide (TC) bit, examination of recovered rock cuttings, and correlation with moisture content tests. The strength of the bedrock which was cored was assessed by examination of the recovered rock core and subsequent correlation with laboratory Point Load Strength Index testing.

Groundwater observations were made during and on completion of augering and coring each individual borehole. A 50mm diameter slotted PVC standpipe was installed in BH101, BH103 and BH105 for long term groundwater monitoring. These three boreholes were subsequently bailed. The groundwater levels were re-measured after bailing and after a further 13 days (up to 15 days after drilling). For further details on the investigation procedure adopted, reference should be made to the Report Explanation Notes.

Our geotechnical engineer set out the borehole locations, nominated the sampling and testing, directed the standpipe installation and logged the subsurface profile. The borehole logs are presented with this report together with a glossary of logging terms and symbols used.

Selected disturbed samples were recovered from the site and returned to Soil Test Services (STS), a NATA registered laboratory, for moisture content, Atterberg Limits, linear shrinkage, compaction and soaked CBR testing. Another NATA registered laboratory, Labpoint Pty Ltd carried out chemical (pH and sulphate content) testing on selected soil samples. The test results are summarised in the attached Tables A and B and on the Labpoint test report. The rock core was also returned to STS, where it was photographed and selected sections of core subjected to Point Load Strength Index testing. The core photographs are attached opposite the relevant borehole log and the Point Load Strength Index tests are indicated on the borehole logs and are summarised in Table C. Contamination screening of the site soils was outside the agreed scope of work.

3 RESULTS OF INVESTIGATION

3.1 Site Description

The surrounding topography is characterised by undulating hilly terrain which falls away to the north and east. The development site is located on the northern side of Rutledge Street and to the east of its intersection with Trelawney Street, on a hill that slopes at approximately 1° to 3° down to the north. The site is also bounded by West Parade to the south-east and to the north by Rowe Street and Rowe Street Mall. In plan, the site consists of a battle axe block incorporating the existing Eastwood Shopping Centre complex and adjacent shops to the west.

The survey plan, which forms the basis for Figure 1, indicates that ground surface levels fall from about RL 75.0m on the footpath adjacent to the south-west corner and RL 73.8m at the south-east corner of the site falling to about RL 69m along the Rowe Street frontage. An elevated concrete ramp provides access from Trelawney Street to the car park above the shopping centre.

At the time of the fieldwork, the site was occupied by several adjoining one and two storey, brick buildings (shops) and an asphaltic concrete paved car park in the western portion of the site, and Eastwood Shopping Centre complex, a multi-level building containing two levels of shopping, with three and four parking levels above the shopping centre. There is also a commercial tower above the shopping centre in the north-east corner on the Rowe Street Mall frontage. Based upon a cursory inspection, the building appeared to be in reasonably good external condition. There are some trees within and adjacent to the site.

The neighbouring properties to the north-east and north-west contain one and two storey brick commercial and retail buildings. Some of these buildings are constructed up to the common site boundary. The buildings appeared to be in good external condition with some minor wall cracks in places. The adjacent single storey brick (church) buildings are located to the south-west, with part of the church hall close to the common boundary.

The adjacent roads are surfaced with asphaltic concrete, generally in good condition. West Parade is a divided road and its upper carriageway slopes down to the north at approximately 2° to 4°; Trelawney Street also slopes down to the north at about 1° to 3°. The lower (West Parade) carriageway, which is located at the toe of a 3m high concrete retaining wall, runs below Rutledge Street and is adjacent to the Main North Railway further to the east. A concrete bridge on Rutledge Street passes over the railway to the south-east of the site. Rowe Street Mall is brick paved with a pergola (arch) type structure at its eastern end.

3.2 Subsurface Conditions

The Sydney Geological Map shows the area to be underlain by Ashfield Shale, black to dark grey shale and laminite bedrock, belonging to the Wianamatta Group. There are no dykes in the immediate site area; however, there is a dyke which runs from the south-west to the north-east, around 1km to the south-west. It is possible that sub-vertical joints run across the site with a similar orientation.

Reference should be made to the borehole logs for specific details of the variable subsurface conditions encountered at each test location. Graphical summaries of the borehole information illustrating the subsurface geology are presented in Figures 2A, 2B and 2C.

In general terms, the boreholes and the previous DP boreholes encountered existing pavements, shallow, and in places, deep fill over residual silty clays and clays, which grade into weathered shale/siltstone bedrock at depths between 5.0m and 7.15m below existing levels. The more pertinent details of the encountered variable subsurface conditions are presented in the following.

- **Asphaltic Concrete** (AC), 50mm thick, was encountered at the surface of BHs 1 and 2. The AC was underlain by 100mm of road base in BH1, and by fill in BH2.
- **Concrete**, 55mm to 250mm thick, was encountered at the surface of BHs 103, 104 and 105. In BH104, silty sandy igneous gravel (base material) was revealed below to concrete overlying silty clay. There was no sub-base material below the concrete in BHs 103 and 105.
- **Fill** was encountered from ground surface in BHs 101 and 102 and below the pavements in BHs 1, 2 and 105 to depths generally between 0.4m and 1.15m or to 3.8m in BH105 below existing levels. The fill comprised silty sandy gravel, silty sand, or silty clay of medium to high plasticity and contained varying amounts of igneous, sandstone and ironstone gravel, slag, brick and metal fragments. Based on SPT blow counts, the fill was assessed to be variably compacted, mostly in the poorly to moderately compacted range, and moderately to well compacted from 1.1m to 2.9m in BH105. The covers the underlying silty clay. Note that the silty clay from 0.4m to 1.2m depth in BH101 may possibly be fill.

- **Silty Clays and Clays:** The residual silty clays and clays were of medium to high plasticity with varying sizes and proportions of ironstone gravel. These clay soils were initially moisture affected and of firm to stiff strength to a common depth of 1.2m in BH101 and BH104, to 1.5m in BH102, and to 1.8m in BHs 1 and 2. Otherwise, the clays were of very stiff to hard strength, with moisture contents generally greater than the plastic limit. The clays were interbedded with shale from 5.0m to 6.76m in BH2. The silty clays and clays graded into shale/siltstone.
- **Weathered Shale/Siltstone Bedrock:** The shale and siltstone was predominantly distinctly to slightly weathered and of medium to high strength. However, poor quality (extremely low strength) rock was penetrated at 6.6m in BH104, 6.7m in BH1, and at 6.8m in BH105. The poor quality shale/siltstone profile contained iron indurated bands. The underlying shale in BHs 104 and 105 and the siltstone on first contact in BH2 was of low or low to medium strength to between 7.6m and 9.2m, then medium or medium to high strength thereafter. Defects within the cored shale/siltstone comprised occasional, extremely weathered seams (between 5mm and 30mm thick), clay seams, or sub-horizontal bedding planes, and several (30° to 90°) joints. The core loss zones in BHs 1, 2, 103 and 105 are inferred to be extremely weathered seams.
- **Groundwater:** The boreholes were 'dry' during and on completion of auger drilling. The use of water within the cored boreholes obscured further measurements of groundwater levels during and after core drilling. On completion of drilling, slotted PVC standpipes were installed at four locations, BHs 2, 101, 103 and 105. Note that groundwater was measured at 4.7m in BH2 about 1 week after drilling; it is not known whether this borehole was bailed after drilling. In the other boreholes, the drilling water was generally bailed from inside the standpipe.

Details of the recently installed standpipes and the subsequent groundwater monitoring are given in Table 1.

Table 1 – Groundwater Monitoring Results

Standpipe Location	Screen Depths below Ground Surface (m)	Bailed Water Depth below Ground Surface (m)	Depths below Ground Surface/Reduced Levels of Groundwater after Bailing							
			26/09/07		27/09/07		28/09/07		9/10/07	
			Depth (m)	Approx. RL (m)	Depth (m)	Approx. RL (m)	Depth (m)	Approx RL (m)	Depth (m)	Approx. RL (m)
BH101	6.2 - 14.16	5.45	1.81	69.6	1.20	70.2	-	-	0.95	70.5
BH103	7.0 - 10.0	2.75	NA	-	NA	-	2.06	68.4	1.80	68.7
BH105	6.3 - 14.3	13.00	13.0	61.3	7.75	66.6	-	-	7.35	66.9

Note that the inflow rate was initially 3 litres per minute into BH101 after bailing reducing to an average of about 0.5 litres per minute over the first hour of monitoring.

3.3 Laboratory Test Results

The moisture content tests on samples of the rock correlated well with our field assessment of rock strength. The approximate Unconfined Compressive Strengths (UCS) of the rock core, as shown on Table C, varied from 6MPa to 38MPa, with an average of about 19MPa.

The results of Atterberg Limits and Linear Shrinkage tests indicated that the indicated the natural silty clay is of high plasticity and has a high potential for shrink/swell movements with changes in moisture content, that is, Class 'H' soils in accordance with AS2870. The four day, soaked CBR value was 3% for the natural silty clay of medium to high plasticity in BH103.

The soil pH test result indicated that the silty clay sample was acidic with a pH value of 5.1, which indicates that some measures should be taken to protect buried concrete in contact with these soils. The sulphate content was less than 50mg/kg.

4 COMMENTS AND RECOMMENDATIONS

4.1 Summary of Principal Geotechnical Issues and Further Work

Based on the results of this preliminary subsurface investigation carried out, the principal geotechnical issues for the development are as follows:

- The proposed development will presumably involve substantial changes to the site including demolition of the existing buildings, retaining walls and other structures, pavements, and excavations of large volumes of soil and rock. Good engineering design, construction and maintenance practices should be adopted to maintain stability to adjoining buildings and structures during excavation and in the long term, as well as reducing the risk of vibrational damage to adjoining buildings and structures during excavation.
- The shallowest groundwater levels were observed within the natural silty clay and clay, possibly as localised flows through permeable gravelly layers, and also from defects within the shale/siltstone above the proposed basement. Hence, excavations are likely to encounter groundwater seepage requiring some dewatering during construction and the provision of drainage behind the permanent basement walls and below its floor. Monitoring of groundwater levels in the standpipes should continue during both the design and construction phases.
- The proposed building of moderate to high loads should be founded on the underlying bedrock. Where bedrock is exposed or at shallow depth after site earthworks, pad or strip footings may be used, but piles will be required where the depth to rock is deeper than about 1.5m.

Further comments on the above and other issues are provided within the following sections of this report are based on seven boreholes distributed throughout the site, where access permitted. Due to the variability encountered already on the site, we recommend that further boreholes be drilled once access is made possible to provide further information on depths to reasonably good quality shale/siltstone around the basement perimeter. This investigation could be better focused on the information required once the design is further advanced and a preferred excavation support and foundation solution is identified. The recommendations given in the following sections may require review and possible amplification, once the design is further advanced. A summary of additional geotechnical work recommended are provided in Section 5. It will be essential during excavation and construction works that regular geotechnical inspections be commissioned to check initial assumptions about excavation and foundation conditions and possible variations that may occur between inspected and tested locations and to provide further relevant geotechnical advice. Irregular or 'milestone' inspections by a geotechnical engineer are often not adequate for excavation, shoring and foundation works. It is recommended that the Client be made aware of the need to commission a geotechnical engineer for regular frequent inspections. The comments provided in this report should be reviewed following these inspections. A meeting of the design team may be of benefit in order to discuss the geotechnical issues and solutions in more detail.

4.2 Dilapidation Survey and Adjacent Buildings

Construction of the proposed basement levels will require bulk excavation to depths of about 6m to 12m below existing levels. The proposed excavation will occupy the whole site. The perimeter of the excavation will be immediately adjacent to the existing buildings to the north-east and west and footpaths in West Parade, Rutledge and Rowe Streets.

Prior to commencement of construction, we recommend that dilapidation surveys be carried out the neighbouring buildings and structures within about 12m to 15m of the excavation perimeter. These surveys would provide a record of existing conditions prior to commencement of the work. A copy of the reports should be provided to the adjoining property owners who should be asked to confirm that they represent a fair assessment of existing conditions. They can also be used for assessment of potential damage claims. These reports should be carefully reviewed prior to excavation commencing; in particular, the size/energy of the impact breakers should be considered. We can complete these dilapidation surveys if you wish to commission us.

During the excavation, every care should be taken not to undermine or render unstable the footings of any adjoining structure.

Details of the neighbouring buildings to the north-east and west are unknown to us and should be checked from available records. Presumably, these one and two storey buildings are supported on conventional high level footings founded in the natural silty clays and clays; however, this should be confirmed by excavating test pits to expose the building footings and their foundations.

4.3 Excavation

4.3.1 Excavation Methods

An assessment of the excavation characteristics of the various strata is presented in the following. The excavatability of the shale/siltstone and the selection of appropriate excavation equipment have been assessed on the basis of the rock core strength and limited information on the nature and inclination of rock defects. Assessment of excavation characteristics and productivity is not an exact science and contractors must make their own evaluation based on experience with specific equipment.

Following demolition of the existing buildings, excavation and removal of pavements, old footings, slabs, services, etc will be required. The underlying fill and soils should be readily excavated by conventional earthworks plant (eg small to medium excavator, dozer blade, etc).

Excavation in very low to low strength shale/siltstone could easily be achieved using a Caterpillar D7 tractor or equivalent, probably with some light to medium ripping. Much of this material can probably also be excavated with a large excavator bucket; localised stronger bands/zones may require heavy ripping or the use hydraulic rock hammers. However, excavation through the shale/siltstone, which is expected to be predominantly of low to medium strength or stronger, will present hard ripping or "hard rock" excavation conditions.

Due to the close proximity of adjoining buildings, our recommendation would be to excavate within the low to medium strength bedrock with rock saws, then by ripping tynes on medium to heavy excavators or dozers. Rock grinder attachments on medium to heavy hydraulic excavators could be used but productivity with rock grinders would probably be low.

We expect that excavation of the medium to high strength or stronger rock will more difficult, requiring large rock saws in combination with heavy ripping using Caterpillar D11 or Komatsu D475A dozers. A generous allowance should be made for rock hammer assistance to the ripping, especially where rock defect spacing (bedding and cross bedding, joints, etc) is greater than about 0.5m. Hydraulic rock breaking equipment would also be required for detailed excavations such as footings or services. An assessment of excavation methods should be made by the excavation contractor, preferably after inspection of the rock cores (we only store these for one month after the formal report is issued unless other arrangements are made). The ease with which excavation of rock is achieved depends upon the equipment used, the skill and experience of the operator and the characteristics of the rock. The contractor must make his own judgement on all of these factors.

4.3.2 Potential Vibration Risks

Use of heavy rock breakers will cause noise and vibrations. Such vibrations should be closely monitored by the site superintendent. We recommend continuous electronic vibration monitoring (i.e. measurement of peak particle velocities) be carried out on this site during the period of excavation. Monitoring points should be set up on adjoining buildings. These monitoring points should have a warning light system incorporated to show when vibrations have exceeded allowable limits. As an initial guide, we recommend that peak particle velocities should not exceed 5mm/sec on adjoining buildings and structures (see attached Vibration Emission Design Goals sheet). This limit of vibrations should be reviewed once dilapidation reports have been completed to confirm that they are still suitable. By monitoring vibrations in this way, it will allow some freedom to the excavation contractor in the equipment he adopts, so that a balance can be made between productivity and vibration reduction.

Vibrations induced by excavations can be reduced by alternative methods such as the following. Due to the depth of excavation and the close proximity of the adjoining buildings, one or more of the following methods will be required during excavation.

- Start the rock excavation away from likely critical areas.
- Maintain rock hammer orientation into the face and enlarge excavation by breaking small wedges off faces.
- Operate hammers in short bursts only, to prevent amplification of vibrations.
- Use smaller equipment (offset by a loss in productivity and economy and greater duration of the nuisance).
- Excavate a cut off trench around the site to reduce vibrations from excavation activities; this can be done progressively with the rock saw.
- Use line drilling, especially along boundaries, to aid breaking and trimming.

As a very general guide, where adjoining buildings are about 1m or so from the boundary, we have found on other sites that grinders or rock saws are typically required within about 3m of the site boundaries. However the distance is very dependent on specific rock characteristics at each site, the equipment used and the condition of adjoining buildings, therefore vibration monitoring is essential.

In addition, we recommend that only excavation contractors with appropriate insurances and experience on similar projects be used. The contractor should also be provided with a copy of this report to make his own judgement on the most appropriate excavation equipment.

4.3.3 Groundwater

All seven boreholes remained 'dry' during and on completion of auger drilling. Subsequent monitoring after bailing of water used during coring, indicated groundwater levels at 0.95m depth (RL 70.5m) in BH101 and 1.8m (RL 68.7m) in BH103, probably through permeable gravelly layers in the natural silty clays, or from defects within the shale bedrock at 7.35m (RL 66.9m) in BH105, which is above the proposed basement level of RL 62.55m. It is recommended that further measurements be made within the standpipes to provide a better assessment of the long term groundwater level fluctuations. Notwithstanding, it is apparent from the groundwater monitoring undertaken so far that drainage will be required both during construction and in the long term behind all retaining walls and below the basement floor slab.

Nevertheless, the subsurface profile as a whole is composed of relatively low permeability strata (residual silty clay and clay and shale/siltstone bedrock). The infiltration rate through the soils and into the proposed excavation is dependent on many factors including subgrade variability (eg gravelly or sandy layers in the clays, old root holes, fissures, etc), the applied head of water, changes in moisture and density throughout the subsoil profile. The excavation in the shale/siltstone would also expose joints, bedding partings and other defects, which we surmise exist, both in the excavated rock face and the basement floor; seepage may flow through these defects, especially during wet weather periods. The groundwater may well be higher and more extensive following heavy rain.

In summary, our opinion is that a conventional sump and pump system is likely to be suitable during construction. Initially, flow through open joints in the rock may be high but these are expected to lessen considerably with time as the surrounding rock mass is drained. We recommend that careful monitoring of seepage be implemented during the excavation works to confirm the capacity of the drainage system. We do not consider that there is a likelihood of the construction of the basement causing significant interference to the groundwater flow nor it being untowardly affected by the groundwater provided proper drainage systems are designed and installed by a qualified hydraulic/drainage engineer.

Given the depth of the proposed excavation, our preference would be to provide a drainage layer (and/or 'rock-saw' slots cut into the shale/siltstone floor) below the basement slab to safeguard against the possibility of flooding and groundwater pressure causing an uplift pressure. This drainage could be incorporated with the wall drainage (if constructed) or perimeter open drains around the basement. As a guide for preliminary budgeting purposes only, allowance should be made for a free draining gravel bed, 300mm thick, with 100mm diameter slotted pipes on say a 4m grid. We recommend that the groundwater management system be designed by the hydraulic/drainage engineer. Note that the groundwater monitoring results are given in Table 1 (in Section 3.2). The drains should incorporate a sump and automatic fail-safe pump system for discharge of collected seepage to the stormwater system. Otherwise the floors would have to be designed for hydrostatic uplift pressures.

Appropriate waterproofing and drainage will be required for the permanent walls in contact with the excavated areas.

4.4 Excavation Support

4.4.1 Batter Slopes and Treatment

Temporary excavations in the fill, soils and any poor quality (extremely low to very low strength) rock will collapse if cut vertically and should be supported by a suitable retention system installed prior to commencing with bulk excavation. It is very unlikely that it will be possible to form temporary batter slopes around the sides of the excavation for the presently proposed layout of the basement.

We would normally recommend that the excavations in the silty clay fill in at least a moderately compacted state, silty clay of at least very stiff strength and extremely low to very low strength shale/siltstone may be battered at 1 Vertical (V) in 1 Horizontal (H). Low strength shale/siltstone may be cut at 1V in 0.75H; batters in stronger rock are discussed in Section 4.4.2.

Surcharge loadings (footings, vehicles, etc) should not be within the zone of influence of the excavation. As a guide, surcharge loadings should be no closer than 2H from the top of any batter or the face of any excavation (including footing excavations), where H is the vertical height of the batter or depth of the excavation in the fill, silty clay and low strength or weaker shale/siltstone.

Flatter batters may be required in the sandy fill and clays of stiff strength or weaker or where groundwater seepage is encountered. Where possible, water should be drained away from batter slopes and prevented from discharging over batter faces.

Permanent batters would need to be flatter (that is, no steeper than 1V in 2H) and protected from erosion by vegetation or other means.

4.4.2 Unsupported Rock Faces and Treatment

Good quality shale/siltstone of at least medium strength may be cut to a temporary batter of about 1V in 0.25H or possibly vertically and the face left temporarily unsupported.

The stability of battered cuts or near vertical cuts, even in good quality, medium strength or stronger bedrock, must be subject to confirmation by an inspection by a geotechnical engineer. No excavation face should be allowed to advance more than 1.5m vertically between inspections and the excavation should be staged or stepped so that a whole face is not excavated 1.5m vertically between visits. If adverse defects are identified by the geotechnical engineer during the inspections, then stabilisation or flatter batters will be required. Such treatment can be necessary due to the presence of adversely orientated defects or zones, which may form continuous planes of weakness, such as inclined joints, affecting the stability of unsupported rock faces. Our assessment of site conditions is based only on the boreholes, which only provide a very limited sample of the shale/siltstone. Our preference would be to commence the rock excavations well away from the perimeter of basement to allow for the identification of potential larger scale instability (continuous joints can be as flat as 40° to 50° and run in north-west/south-east or north-east/south-west directions) that occasionally exists within shale/siltstone bedrock. Should these joints exist, flatter batters (possibly of the order of 1V in 1H or flatter) or large capacity rock anchors can be required; the cost of the latter would be relatively high and delays to the excavation process with consequential cost implications would occur.

As there appears to be only occasional defects in the medium to high strength shale/siltstone, the face may be protected by dowels, mesh and shotcrete or the permanent basement walls. The extent of shotcrete to temporarily protect the rock faces prior to construction of the permanent walls should be confirmed during the geotechnical inspections. Stabilisation may also require the use of rock bolts, mesh and/or shotcrete protection to support the large blocks or other rock face areas. It would be unusual to complete such an excavation without some form of support being required to the rock faces, though this may take forms other than rock bolting.

The permission of adjoining property owners, to install rock bolts below their property, should be obtained in advance of construction in order that there is no delay in providing support should adverse conditions be encountered.

Any potentially unstable blocks in the exposed shale/siltstone face should be clearly identified with paint markings so that proposed remedial work can be easily communicated to, and undertaken, by the contractor. We expect that face works probably need to include the following:

- Scaling down of some small blocks using a crowbar etc.
- In critical areas of the rock face, localised support (eg shotcrete, rock bolts, dowels, buttresses) may be required.
- Mesh may need to be draped and fixed to the face to prevent small blocks from falling and endangering site personnel during construction.

A moderate provision for rock bolting and shotcrete and mesh should be included in the Contract Documents. These works can be nominated following the geotechnical inspections. For preliminary budgeting purposes, our best guess would be to allow for installing rock bolts, on say a 2m grid, to support around 10-15% of the exposed rock face. The rock bolts would possibly be 2m to 5m long hot dipped galvanised 24mm diameter bars, installed in 75mm drill holes, threaded at the heads and fitted with galvanised square anchor plates, spherical washers and nuts. Other measures could be required.

If permanent rock-bolts are to be avoided, then stabilisation works must be designed such that propping from the main structure is possible as the construction progresses.

Where space permits, the retaining walls would then be constructed at the toe of the temporary batters or vertical cuts and subsequent backfilling undertaken. Caution will be required during backfilling to prevent over compaction adjacent to retaining walls and thereby causing excessive forces on the walls.

4.4.3 Shoring Systems and Retaining Walls

A suitable method of retention to support vertical cuts, prior to bulk excavation, would be bored cast in-situ or augered, grout injected (CFA), soldier pile walls with infill panels where movement is not of concern, or alternatively, contiguous pile walls. Construction of the contiguous pile walls should be of high quality, taking the uttermost care to prevent soil loss through gaps that may occur between the piles as this would add to the possibility of settlement occurring outside the excavation. Such gaps should be rectified without delay, such as by mass concrete infill.

Conventional driven sheet-pile walls would not be suitable as there is a need to minimise noise and avoid ground vibration damage to the neighbouring buildings.

We advise that cantilevered walls may be used for supporting retained heights of around 3m to 4m and only where some higher lateral and vertical movements of adjoining ground can be tolerated. If greater height walls are required, or, where only minimal movements can be tolerated, then anchored or propped walls would normally be required.

All soldier piles of the shoring walls should be taken down and embedded below the basement excavation. The piles of the contiguous pile walls should be founded at least 0.3m into shale/siltstone of medium to high strength and below this it would generally be possible to excavate an unsupported vertical rock face, provided at least some piles in contiguous pile walls (eg. at say around 2.5m to 3m intervals) are taken down below the basement excavation. The magnitude of building loads carried by the perimeter walls may affect the quality of the rock on which the shoring must be founded.

Anchors or props will be required at the base of the walls founded above a vertical shale/siltstone rock face to provide the required lateral stability; the construction sequence must be carefully evaluated to ensure adequate horizontal restraint throughout the process. A 'hold' point on excavation should be implemented once the bases of the retaining piles are reached to allow inspection of the rock by a geotechnical engineer prior to continuing with the bulk excavation. Poorly constructed piles or piles terminated on poor quality rock would require underpinning.

Care must be taken during rock excavation not to over-excavate the rock foundation supporting the toe of the shoring system. It is highly desirable for there to be an offset to the cut face below the piles to give some "construction tolerance".

Props or anchors will also be needed to restrain the upper sections of the walls and these must be installed progressively and immediately once the propping point has been uncovered, and prior to excavation adjacent to neighbouring structures and sensitive services which are located within the 2H zone of influence of the excavation perimeter (discussed in Section 4.4.1).

Drilling of rock sockets will be difficult through the iron indurated bands and medium to high strength rock requiring the use of heavy drilling rigs equipped with rock augers. Some groundwater inflow is expected into bored pile footings and we expect that this inflow will be controllable by conventional pumping methods. Alternatively, concrete may be poured using tremie methods.

Along the Rutledge Street frontage, it may be possible to retain the existing basement walls to maintain support for the footpath and road, if they are founded on competent shale/siltstone. However, details of these basement retaining walls and footings are unknown to us and must be checked both for strength and stability, by the structural engineer. This would require a review of available records or alternatively, test pits should be dug to expose the existing footings and on what they are founded. This review should be carried out prior to finalisation of the structural drawings. If not founded on competent rock, the existing wall should be underpinned to found in the underlying shale/siltstone of at least medium strength and of adequate bearing capacity. The underpinning may be installed into vertical slots of 1m to 1.2m maximum width cut into the rock. We recommended that the construction of the underpinning be carefully sequenced using hit-miss-construction techniques. The underpinning should be designed for lateral earth pressures, any surcharge loadings and hydrostatic pressures. It may well also be necessary to provide lateral restraint in the form of props or anchors if the ground in front of the wall is removed; the strength and stability of the existing wall and the need for anchors/props should be checked by the structural engineer.

Detailed construction supervision, monitoring and inspections will be required during the piling and subsequent excavation.

We recommend that only experienced contractors be considered for excavation works and wall construction.

4.5 Lateral Earth Pressures

Design of cantilever retaining walls may be on the basis of an 'active' lateral pressure coefficient, K_a, of at least 0.35 for the fill, clayey soils, extremely low strength shale/siltstone, provided some deflection is tolerable. The K value may be reduced to 0.2 for shale of at least very low strength rock. A bulk unit weight of 20kN/m^3 for the soils and 22kN/m^3 for any extremely low to very low strength rock may be adopted. Walls which are to be subsequently propped by the permanent structure (e.g. by the upper ground floor slab) should be designed based on a higher lateral pressure coefficient, K, of at least 0.6 (or 0.4 for very low strength rock). The good quality shale/siltstone of at least low strength can be taken to be self-supporting and no 'K' values need to be taken into account; this should be confirmed during geotechnical inspections during construction. These coefficients assume almost horizontal ground surfaces behind the crest of the walls.

For propped or anchored walls, we recommend the use of a trapezoidal lateral earth pressure of at least 4H (kPa), where H is the retained height in metres in the soils and poor quality shale/siltstone. For propped or anchored walls in areas, which are highly sensitive to lateral movement (such as adjacent to neighbouring building footings located within 2H metres of the excavation), a trapezoidal lateral earth pressure of at least 8H (kPa) should be used. These 4H and 8H pressures should be assumed to be uniform over the central 50% of the full, retained height in the soils and poor quality shale/siltstone. Alternatively, more sophisticated computer based shoring design (such as Wallap) generally results in cost savings compared to designs based on simplified assumptions regarding earth pressure distributions. These detailed numerical analyses can model the progressively anchored or propped shoring walls as they are constructed. The geotechnical design parameters for the various strata nominated in Table 2 may be adopted to confirm the minimum depth of embedment of the wall toe and the likely order of magnitude of wall movements during the various phases of construction when using Wallap.

Table 2 – Wallap Wall Design Parameters

Strata	Bulk Unit Weight (kN/m^3)	Undrained Cohesion (kPa)	Angle of Internal Friction (degrees)	Poisson's Ratio	Elastic Modulus (MPa)
Existing Fill – silty sand, silty clay assessed to be poorly to moderately compacted	18	0	25	0.3	5 – 8
Silty Clay/Clay – firm to stiff	18	30 – 50 (0 - 5)	0 (20)	0.35	5 – 10
Silty Clay/Clay – stiff	19	80 – 100 (0 - 5)	0 (20 – 25)	0.25	10 – 15
Silty Clay/Clay – very stiff	20	150 – 180 (0 – 10)	0 (25 – 30)	0.25	20 – 25
Silty Clay/Clay – hard	20	200 – 250 (0 – 10)	0 (25 – 30)	0.25	30 – 35
Shale/Siltstone – extremely low strength with iron indurated bands	22	300	35	0.25	100 – 150
Shale/Siltstone – low strength	23	1000	40	0.25	400 - 500
Shale/Siltstone – low to medium strength	23	1500	40	0.2	500 – 700
Shale/Siltstone – medium or medium to high strength	23	3000	45	0.2	1000 – 2000

Note – for undrained analysis, adopt modulus values towards the higher end of the range for the clays.
– for long term conditions, adopt bracketed values of cohesion and friction.

The recommended lateral earth pressure coefficients and trapezoidal pressures assume almost horizontal ground surfaces behind the crest of the walls. If inclined backfill surfaces are to be designed, then the above factors would have to be increased or the inclined section of backfill should be taken as a surcharge load in the design.

Applicable hydrostatic pressures should be added to the lateral earth pressures, unless specific measures are taken to introduce complete and permanent drainage of the ground behind the walls. Any surcharge affecting the walls (e.g. footings, retaining walls and their backfill, the ground slope behind the wall, etc.) should also be considered in the design.

Lateral toe restraint can be calculated using a triangular earth pressure distribution with a 'passive' earth pressure coefficient, K_p, of 3 for silty clay and clay of at least very stiff strength (but with a factor of safety of at least 2 to limit deformations), assuming horizontal ground in front of the wall. The passive pressure due to the upper 0.3m below bulk excavation level should be ignored in the analysis to take excavation tolerances into account.

For wall footings fully embedded into the underlying shale/siltstone bedrock below the building basement floor level, an estimated allowable lateral toe resistance of 350kPa may be adopted for rock of at least low strength. The lateral stress may be increased to 1200kPa for the medium strength or stronger shale/siltstone. These passive resistance values assume excavation is not carried within the zone of influence of the wall toe and the rock does not contain unfavourable defects etc. The upper 0.3m depth of the socket should not be taken into account to allow for disturbance effects during excavation.

Anchors bonded into at least medium strength shale/siltstone bedrock, may be designed based on a maximum allowable bond stress of 400kPa. All anchors should be proof loaded to at least 1.3 times their working load. Anchors must be bonded behind a 45° line drawn upwards from the base of the excavtion. Anchor group interaction must also be taken into account. Permanent anchors should have appropriate corrosion provisions.

4.5.1 Excavation Induced Movements

It is inevitable that the excavation will induce movements of the adjacent ground that falls within the area of influence of the excavation.

Lateral and horizontal movements could occur within about 2H back from the anchored/propped wall. With a less rigid support system, excavation induced movements should be expected to be of a higher order. Settlements may also be caused by the wall construction itself (e.g. loss of ground during anchor drilling, etc).

As excavation of the rock progresses, the rock mass will also tend to move inwards towards the excavation along bedding planes, clay seams, etc. as it is stress relieved. With increasing depth of excavation, the bed undergoing excavation will also drag overlying beds with it as the lower bed moves towards the excavation. The extent of movement will depend on the strength of the rock between the bedding planes and the spacing of joints or other defects. As the beds move inwards, joints, etc. will start opening behind the excavated face and any structures on or in the rock also move. These stress-relief movements will decrease away from the excavated face, however, their magnitude will increase as the depth of excavation increases.

Experience with excavations in residual clay and poor quality (extremely low to very low strength shale/siltstone indicates that lateral and vertical ground movements of around 2 to 5mm/m of excavation depth may occur, mostly as a result of stress relief, and depending on the rigidity and construction practice of the shoring system. In the medium strength shale/siltstone, these movements may be of the order of 1mm/m of excavation depth. The extent of influence may be defined as extending a horizontal distance from the excavation equal to at least twice the excavation depth.

It may not be practicable to prevent significant vertical and lateral ground displacements immediately beyond the limits of the excavation, so the effects of the inevitable excavation induced movements on the adjoining buildings and structures and also on the permanent structure should be assessed.

The objective with properly engineered retaining walls is to keep the adjacent ground movements within tolerable limits. The actual wall movements are highly dependent on the construction sequence, detailing and quality of installation, and should be closely monitored in critical areas. The extent of significant influence can be defined as extending a horizontal distance from the excavation perimeter equal to twice the excavation depth. Hence, any existing adjoining structures, or buried services, which fall within this area of influence of the excavations, should be assessed for risks of damage due to excavation-induced movements and whether underpinning is required.

It is likely that neighbouring buildings are founded on high level footings but this has not been confirmed. If this is found to be the case, the footings may need to be underpinned down to the underlying weathered shale/siltstone of at least low strength (but not less than that required to support the underpinning loadings) prior to construction of the contiguous pile wall. We recommended that the construction of the underpinning be carefully sequenced and constructed progressively, if necessary using hit-miss-construction techniques. The underpinning should be designed for lateral earth pressures, any surcharge loadings and hydrostatic pressures.

The risk of architectural or structural damage to adjoining buildings will depend on their sensitivity to horizontal and vertical deformations, structural load, type and founding elevations of the floor slabs and footings and foundation conditions. All these factors should be carefully investigated and evaluated prior to excavation commencing.

In addition, we recommend that an excavation/retention methodology be prepared prior to bulk excavation commencing. The methodology must include but not be limited to proposed excavation, retention and underpinning techniques, the proposed excavation equipment, excavation/retention/underpinning sequencing, geotechnical inspection intervals or hold points, vibration monitoring procedures, monitor locations, monitor types, contingency plans in case of non-compliance. Preferably, this methodology should be shown on the structural engineer's drawings. The excavation/retention/underpinning methodology should be reviewed and approved by the geotechnical engineer.

4.6 Footings

Following bulk excavation, shale/siltstone will be exposed over most of the basement. It is recommended that all footings for the building and retaining walls be founded within the shale/siltstone to provide uniform support and reduce the potential for differential footing settlements. Pad and strip footings or bored (cast-in-situ or augered grout) piles may be designed for the maximum allowable working end bearing pressures at the indicative founding levels tabulated in Table 3. Rock sockets below the indicative founding levels specified in Table 3 may be designed for safe adhesion values of 10% of the appropriate safe bearing pressure under compressive vertical loading (ie provided excavation is not carried out within the zone of influence of the footing).

Table 3 – Footing Bearing Pressures and Depth

Borehole Number	Depth below Existing Ground Level/Reduced Level for Safe Bearing Pressure of 1.0MPa		Depth below Existing Ground Level/Reduced Level for Safe Bearing Pressure of 3.5MPa		Depth below Existing Ground Level/Reduced Level for Safe Bearing Pressure of 6.0MPa	
	Depth (m)	Approx. RL (m)	Depth (m)	Approx. RL (m)	Depth (m)	Approx. RL (m)
1	7.3	66.8	7.3	66.8	8.0	66.1
2	7.1	66.7	7.7	66.1	10.1	63.7
101	5.3	66.1	6.5	64.9	6.8	64.6
102	7.2	62.4	7.2	62.4	7.2	62.4
103	7.0	63.5	9.0	61.5	9.0	61.5
104	6.9	63.1	8.2	61.8	8.2	61.8
105	7.0	67.3	8.6	65.7	10.2	64.1

Where the footings are founded close to a steeply inclined or vertical rock face, the allowable bearing pressure below these footings will need to be carefully assessed. The safe bearing pressure would need to take into account rock strength, bedding, jointing and the influence of clay seams as well as the magnitude and inclination of the applied loadings. Inspections by a geotechnical engineer of each exact, footing affected rock area should be requested to assess its quality and make a judgement on final design pressure.

If the designer wishes to adopt the limit state design methods, such as in the Piling Code, AS2159-1995, then the ultimate values of end bearing pressure and lateral stress (refer to Section 4.5) may be estimated by multiplying the recommended allowable values in Table 3 by Factors of Safety of 3. A Factor of Safety of 2 should be applied to the shaft adhesion values. We recommend that the ultimate values be multiplied by a geotechnical strength reduction factor, Φ_g, of 0.5. Higher reduction factors may be adopted but these will depend on the intensity and type of proving of the footings and their foundation. Appropriate load factors should also be applied to the proposed footing loadings.

The allowable bearing pressures given in Table 3 are based on a serviceability criteria of deflections at the footing base/pile toe of less than or equal to 1% of the least footing dimension (or pile diameter). Footings on rock can also be designed using 'Limit State Design' principles as detailed in the paper "Foundation on Sandstone and Shale in the Sydney Region' by Pells, Mostyn and Walker, Australian Geomechanics, Number 33, Part 3, December 1998 (Pages 17-29). It must be emphasised that the use of limit state design to adopt relatively high bearing pressures (above the serviceability criteria described above) is not currently standard practice, and there is an increased risk of inadequate footing performance.

For earthquake design (in accordance with AS1170.4), we recommend an acceleration coefficient of 0.08 and a site factor of 0.67 be adopted as the building will be founded in shale/siltstone bedrock of low strength or stronger.

We recommend that all footing excavations be checked and approved prior to concrete being poured.

In addition to inspection, the sandstone foundation under footings designed using a safe bearing pressure of 6MPa or higher would require additional proving. At least 30% of footing locations should be spoon tested by a geotechnical engineer or core drilled. The testing of individual footing locations should be carried out to a depth of at least 1.5 times the footing width or 2.5m, whichever is the lesser, below the footing base to confirm that defects are within allowable limits. The presence of significant defects would require a reduction in the allowable bearing pressure or an increase in founding depth. For spoon testing of bored piles, personnel must be able to safely descend the pile excavation, which means that appropriate casing and other safety equipment would be necessary, together with a minimum pile diameter of 900mm. Personnel would also require appropriate training to work in such locations. If this is not possible then we recommend the core drilling of the foundation rock prior to footing excavation, using truck mounted rig equipment with appropriate diamond coring equipment.

If bearing pressures of the footings were limited to 3.5MPa then only inspection by a geotechnical engineer of the exposed rock foundation would be required (i.e. no core drilling or spoon testing at individual footing locations). We can assist with future geotechnical inspections if you wish to commission us at the appropriate time.

For piles socketed into the shale/siltstone, we recommend large capacity drilling rigs with rock drilling equipment be used to drill the piles. The proposed piling contractor must therefore be given a copy of this report to ensure that appropriate equipment with sufficient power is brought to site. Piles should be poured immediately or at the very latest, on the same day, as drilling, cleaning and inspection. Special tools should be used to roughen the sides of load bearing pile sockets in the rock. Some groundwater seepage can be expected during the construction of piers and we recommend that trials should be undertaken to confirm piers can be successfully constructed at the site, otherwise augered, grout injected piles should be used. Piers should be dewatered (by conventional pumping methods) prior to concreting or the concrete may be poured using tremie methods.

The initial stages of footing excavation/drilling, particularly if bored piles are adopted, should be inspected by a geotechnical engineer/engineering geologist to ascertain that the recommended foundation material has been reached and to check initial assumptions about foundation conditions and possible variations that may occur between borehole locations. The need for further inspections can be assessed following the initial visit.

4.7 Basement Car Park Slab

Shale/siltstone is anticipated to be exposed over most of the proposed basement excavation and no special treatment is generally required other than the removal of loose and softened material. Areas, which have to be built-up to infill low points in the excavation should be filled with properly compacted sub-base material (see Section 4.8.2). Although we expect that some under-floor drainage will be required, this should be reviewed following further monitoring of groundwater seepage during and on completion of the excavations. The under-floor drainage (such as perimeter drains, 'rock-saw' slots cut into the shale/siltstone floor and/or a free draining gravel bed) should be installed with sumps for gravity or automatic pumped discharge of groundwater. If under-floor drainage is not installed, then the on-ground floor slab will be subjected to uplift pressures from the groundwater; this may require additional mass or ground anchors.

If natural silty clays and clays are encountered at basement level, for example, possibly towards its north-east corner, the basement floor slab may be designed as a fully suspended structure founded on the shale/siltstone or alternatively, it may be supported on the clayey subgrade; the latter is discussed further in Section 4.8. Note that sections founded on the clay must be separated form sections founded on the shale/siltstone.

The basement floor slab, where subject to traffic loadings, should have a sub-base layer of at least 100mm thickness of crushed rock to RTA QA specification 3051 (1994) unbound base material (or equivalent good quality durable fine crushed rock) which is compacted to at least 100%SMDD.

4.8 Floating Slabs and Pavements

The on-ground floor slab for the buildings and pavements may be founded on the engineered fill or the proof rolled clayey subgrade on condition that the subgrade is prepared in accordance to the recommendations provided in Section 4.8.1 and 4.8.2. The design of pavements will depend on subgrade preparation, subgrade drainage, the nature and composition of new fill imported to the site, as well as vehicle loadings and use.

Lightly loaded "floating" on-ground floor slabs (with floor loadings less than 5kPa) and pavements may be designed using a lower bound characteristic CBR value of 3% or a coefficient of subgrade reaction of 30kPa/mm (750mm plate) or a long term Young's modulus of 15MPa for the proof rolled and/or treated clayey subgrade, which is prepared in accordance with recommendations given in Section 4.8.1.

On-ground floor slabs for the proposed building founded on the proof rolled and treated residual silty clays and clays should be incorporated in a stiffened slab or raft footing system designed to allow for movements in any underlying fill or silty clays and clays, which will generally have a high shrink/swell potential "Class H" in accordance with AS2870. Slabs constructed over the engineered fill or clayey subgrade must be isolated from slab sections founded on the shale/siltstone. On-ground floor slabs subject to traffic loadings should be supported on a sub-base layer of RTA Specification 3051 unbound or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD.

We have not carried out a traffic survey, however, vehicular traffic is presumably to comprise mainly cars and delivery trucks; occasional garbage trucks, etc may also use the area. Table 13.7.3 of APRG Report No. 21 (1998) "A Guide to the Design of New Pavements for Light Traffic" recommends a design traffic loading of 3.5 X 10^5 ESAs for roads (with an Annual Average Daily Traffic of 1200 with up to 6% of vehicles with a gross vehicle mass in excess of 3 tonnes) for the traffic over the 40 year (assumed) design life. Therefore, if the traffic intensity is higher than allowed for, then the life of the pavement would be reduced. This adopted ESA is indicative only and should be confirmed by the traffic engineer. It should also be modified to allow for the effects of heavy forklifts and delivery trucks driving, turning and manoeuvring.

Figure 13.8.2 (A) of the APRG report indicates that a total pavement thickness of around 520mm would be required for this assumed design traffic loading of 3.5 X 10^5 ESAs and CBR value of 3%. This ESA would result in the following pavement layer thicknesses:

	40mm Asphaltic Concrete, compacted in two layers
over	150mm Base Class DGB20
over	330mm Sub-base Class DGS20 or DGS40
over	silty clay subgrade

All upper (base) course should be crushed rock to RTA QA specification 3051 (1994) unbound base and compacted to at least 100% of Standard Maximum Dry Density. All lower (sub-base) course should be crushed rock to RTA QA specification 3051 (1994) unbound base or ripped/crushed sandstone with CBR greater than 40% maximum particle size of 60mm, well graded and Plastic Index less than 10. All lower course material should be compacted to an average of no less than 100% of SMDD, but with a minimum acceptance value of 98% of SMDD.

For rigid pavement design, Table 13.9.5 of APRG report gives a concrete pavement thickness of 210mm for concrete with a flexural strength of 3.5MPa, an assumed equivalent design traffic of 1.5 X 10^6 CVAGs, and a CBR of 3%. Concrete pavements should be supported on a sub-base layer of at least 100mm of RTA Specification 3051 unbound base or equivalent good quality crushed rock, compacted to a density of at least 100% SMDD.

Pavement levels will need to be graded to promote rapid removal of surface water so ponding does not occur on the surface of pavements.

Concrete pavements should be provided with effective shear connection at joints by using dowels or keys. Concrete pavements should preferentially be used in areas where heavy vehicles manoeuvre such as garbage bin and truck unloading areas.

For flexible pavements, in-situ lime stabilisation of the clayey subgrade could be undertaken to reduce total pavement thickness. Alternatively, an appropriate select fill layer comprising good quality well-graded granular material may be used below the pavement.

Improvement of the subgrade CBR design value and consequent reduction of the crushed rock pavement thickness may be achieved by stabilising the clay subgrade with lime to a minimum depth of say 200mm to 300mm. To determine the optimum lime addition rate to achieve the beneficial effect desired, laboratory tests should be carried out. However, an indicative proportion to achieve a CBR of 6% would probably be the addition of 4% of quick lime by dry weight of the clay. The lime must be thoroughly mixed with the clay using specialist blending machines and then compacted to not less than 98% SMDD at ±2% of SOMC.

Only contractors experienced with lime stabilisation should be used. We note that use of lime close to pedestrian and adjacent building areas is generally not preferred unless an acceptable method of dust suppression can be adopted.

Subsoil drains should be provided on the uphill side and along the perimeter of pavements, with inverts not less than 0.3m below clay subgrade level. The drainage trench should be excavated with a longitudinal fall to appropriate discharge points so as to minimise the risk of water ponding. The pavement subgrade should be graded to promote water flow or infiltration towards subsoil drains.

4.8.1 Proof Rolling and Filling

Should any large trees require removal, we recommend they be removed well in advance of construction to allow for readjustment of the moisture content of the highly plasticity (reactive) silty clay subsoil. Removal of any large trees should also include the removal of the tree stumps.

Following demolition and tree removal, subgrade preparation for the proposed building area will require clearance of any other vegetation followed by stripping of root affected topsoil. These materials may be stockpiled or taken off-site as they are not suitable for re-use as engineered fill.

Any existing fill, which is encountered (including backfill to the existing service trenches) is likely to be variably compacted and should be (excavated if necessary) and re-compacted where floor slab support is required. Any remaining existing fill may be left in place below proposed pavements on the condition that the subgrade is proof rolled and appropriately treated. However, there is a chance that some settlement may still occur under pavements bearing on the existing fill, even after it is treated by proof rolling.

Following stripping and excavation to the proposed design levels, the exposed silty clay and clay subgrade should be proof-rolled (say with a 5 tonne minimum deadweight smooth drum roller). Proof-rolling should be carried out under the direct supervision of an experienced geotechnical engineer or geotechnician to assist in the detection of unstable areas which were not disclosed by this investigation. During proof-rolling care should be taken to avoid vibration damage to any neighbouring structures or services or improvements. The vibrations should be monitored and the vibrations may need to be reduced or ceased if there is a risk of damage.

Where unstable areas are encountered the area should be locally excavated down to a sound base and replaced with engineered fill as detailed in Section 4.8.2.

We expect that at present some sections of the exposed subgrade will comprise silty clays with an in-situ moisture content higher than the plastic limit or have been allowed to become wet due to poor site drainage or prolonged exposure to wet periods. These silty clay subgrades may deflect significantly under proof rolling, may exhibit poor trafficability and would not be suitable for construction of new pavements or as a foundation to support slabs in their present condition. It will therefore be necessary to over-excavate such areas to below the depth of moisture 'softening' and to replace the excavated material with properly compacted engineered fill.

Allowance should be made for either tyning, aerating and drying of the subgrade after over-excavation; or lime to dry out and stabilise the subgrade, or for the use of a heavy grade geogrid/geotextile fabric to act as a bridging and separation over the excavation before placing and compacting the engineered fill. Inspection of the excavated subgrade should be undertaken by a geotechnical engineer to confirm the most appropriate method of treatment.

If 'dry' conditions prevail at the time of construction, the subgrade may become desiccated or have shrinkage cracks prior to sealing with sub-base or base materials. If this occurs then the subgrade must be watered and rolled until the cracks disappear.

A poorly drained clayey subgrade will also become untrafficable when wet. We recommend that if soil 'softening' occurs, the subgrade be over-excavated to below the depth of moisture 'softening' and that the excavated material be replaced with engineered fill, compacted as specified in Section 4.8.2.

In addition, in order to improve trafficability for construction equipment, it may be desirable that a capping layer of granular material be placed over any heavily trafficked areas. This granular material (e.g. DGS40 or DGB20) could then be utilised as a sub-base for proposed slab and pavements if desired, though some repair work may be necessary depending on the condition of the layer immediately prior to pavement construction.

4.8.2 Engineered Fill and Compaction Control

Engineered fill should preferably comprise well-graded granular material (ripped or crushed sandstone or ripped shale), free of deleterious substances and having a maximum particle size of 75mm. The sandy fill may be re-used, however, clayey soils are less desirable but may be re-used provided unsuitable ('over-wet' and 'over-size') material and any deleterious material is excluded. The well-graded granular fill for backfilling excavations or for raising site levels should be compacted in layers of not greater than 200mm (or 100mm if hand operated compactors are used) loose thickness, to a density between 98% and 102% of Standard Maximum Dry Density (SMDD). Clayey fill should be compacted to a similar density but within ±2% of Standard Optimum Moisture Content (SOMC). However, it would be wise to have a capping layer of better quality imported fill over the clayey fill. The use of clay materials for engineered fill will entail more rigorous earthworks supervision and compaction control.

Density testing should be carried out at a frequency of at least one test per fill layer per 500m^2 or three tests per layer per visit, whichever requires the most tests.

At least Level 2 testing (or Level 1 where fill is to support movement-sensitive floor slabs/pavements) of earthworks should be carried out in accordance with AS3798. Preferably, the geotechnical testing authority should be engaged directly on behalf of the client and not as part of the earthworks contract. We can complete these tests if you wish to commission us.

The earthworks recommendations provided here should be complemented by reference to AS3798.

4.9 Soil Aggression

The soil chemical tests have revealed acidic subsoil conditions (pH value of 5.1) with sulphate contents of less than 50mg/kg. The designer is referred to the guidelines given in the Cement and Concrete Association Technical Note 57 for appropriate precautionary measures. This document recommends the use of denser concrete mixes to reduce leaching of the cement matrix and additional protection for concrete exposed to soil with a pH value between 4.5 and 5.5. As the pH is relatively low, we also recommend that the cover to steel reinforcement be at least 50mm.

5 SUMMARY OF FURTHER GEOTECHNICAL WORK

Excavation and retention recommendations provided in this report should be complemented by reference to the Code of Practice Excavation Work, Cat. No. 312 by WorkCover NSW.

As detailed in this report, further geotechnical work is recommended as follows:

- Dilapidation surveys for the neighbouring buildings/structures.
- Assessment of the effects of excavation on the nearby building footings and whether underpinning is required.
- Assessment of the effects of excavation on the existing basement wall along Rutledge Street if it is to be retained as temporary shoring and whether underpinning, anchors or props are required.
- Additional boreholes located along the line of the basement perimeter could be considered to provide more extensive information to tenderers for construction of the retention system.
- Additional cored boreholes should also be carried out to prove in more detail the higher bearing capacity (3.5MPa and 6MPa) shale/siltstone strata across the site.
- Quantitative monitoring of transmitted vibrations during rock excavation using rock hammers.
- Assessment of groundwater inflow to confirm drainage requirements following excavation. We also recommend further on-going monitoring of groundwater levels in the standpipes.
- Inspect soldier pile or contiguous pile construction.
- Inspect the rock face at the toes of contiguous piles and the unsupported rock faces in the medium or higher strength rock to confirm batter treatment.
- Inspection of footing excavations to ascertain that the recommended foundation has been reached and to check initial assumptions regarding foundation conditions and possible variations that may occur.

- Inspect proof rolling of silty clay subgrade to detect soft spots requiring treatment.
- Carry out laboratory tests to establish the optimum lime addition rates for pavement/floor slab subgrades.
- This investigation has been limited to boreholes spread throughout site and where access permitted. Additional boreholes may need to be drilled to address particular design issues once design work is commenced and to provide a better coverage across the proposed building and to confirm the variation in depth to rock, and rock quality, especially if bored piers are adopted. For example, where it is proposed to adopt the 6MPa bearing pressure, cored boreholes would be required.

We recommend that Jeffery & Katauskas Pty Ltd view the proposed earthworks and structural drawings and section details in order to confirm they are within the guidelines of this report.

6 GENERAL COMMENTS

The recommendations presented in this report include specific issues to be addressed during the construction phase of the project. As an example, special treatment of soft spots may be required as a result of their discovery during proof-rolling, etc. In the event that any of the construction phase recommendations presented in this report are not implemented, the general recommendations may become inapplicable and Jeffery and Katauskas Pty Ltd accept no responsibility whatsoever for the performance of the structure where recommendations are not implemented in full and properly tested, inspected and documented.

The long-term successful performance of floor slabs and pavements is dependent on the satisfactory completion of the earthworks. In order to achieve this, the quality assurance program should not be limited to routine compaction density testing only.

Other critical factors associated with the earthworks may include subgrade preparation, selection of fill materials, control of moisture content and drainage, etc. The satisfactory control and assessment of these items may require judgement from an experienced engineer. Such judgement often cannot be made by a technician who may not have formal engineering qualifications and experience. In order to identify potential problems, we recommend that a pre-construction meeting be held so that all parties involved understand the earthworks requirements and potential difficulties. This meeting should clearly define the lines of communication and responsibility.

Occasionally, the subsurface conditions between the completed boreholes may be found to be different (or may be interpreted to be different) from those expected. Variation can also occur with groundwater conditions, especially after climatic changes. If such differences appear to exist, we recommend that you immediately contact this office.

This report provides advice on geotechnical aspects for the proposed civil and structural design. As part of the documentation stage of this project, Contract Documents and Specifications may be prepared based on our report. However, there may be design features we are not aware of or have not commented on for a variety of reasons. The designers should satisfy themselves that all the necessary advice has been obtained. If required, we could be commissioned to review the geotechnical aspects of contract documents to confirm the intent of our recommendations has been correctly implemented.

The offsite disposal of soil will most likely require classification in accordance with the Department of Environment & Conservation (NSW) guidelines as inert, solid, industrial or hazardous waste. We can complete the necessary classification and testing if you wish to commission us. As testing requires about seven days to complete, allowance should be made for such testing in the construction program unless testing is completed prior to construction. If contamination is found to be present then substantial further testing and delays should be expected. We strongly recommend this issue be addressed prior to commencement of excavation on site.

If there is any change in the proposed development described in this report then all recommendations should be reviewed.

This report has been prepared for the particular project described and no responsibility is accepted for the use of any part of this report in any other context or for any other purpose. Copyright in this report is the property of Jeffery and Katauskas Pty Ltd. We have used a degree of care, skill and diligence normally exercised by consulting engineers in similar circumstances and locality. No other warranty expressed or implied is made or intended. Subject to payment of all fees due for the investigation, the client alone shall have a licence to use this report. The report shall not be reproduced except in full.

Should you have any queries regarding this report, please do not hesitate to contact the undersigned.

Tony Walker
Associate

QA Review by:

Fernando Vega
Senior Associate
For and on behalf of
JEFFERY AND KATAUSKAS PTY LTD.

115 Wicks Road
Macquarie Park, NSW 2113
PO Box 976
North Ryde, Bc 1670
Telephone: 02 9888 5000
Facsimile: 02 9888 5001

SOIL TEST SERVICES
ABN 43 002 145 173

Ref No:21570V
Table A: Page 1 of 1

TABLE A
SUMMARY OF LABORATORY TEST RESULTS

AS 1289	TEST METHOD	2.1.1	3.1.2	3.2.1	3.3.1	3.4.1
BOREHOLE NUMBER	DEPTH m	MOISTURE CONTENT %	LIQUID LIMIT %	PLASTIC LIMIT %	PLASTICITY INDEX %	LINEAR SHRINKAGE %
101	0.50-0.95	33.6	52	21	31	15.0
101	5.50-6.00	9.3				
101	6.20-6.65	6.9				
104	6.00-6.02	9.4				
104	7.00-7.30	7.9				
105	6.00-6.35	14.4				
105	7.00-7.55	10.7				

Notes:

- The test sample for liquid and plastic limit was oven-dried(50^{O}C) & dry-sieved
- The linear shrinkage mould was 125mm
- Refer to appropriate notes for soil descriptions

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SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 21570V
Table B: Page 1 of 1

TABLE B
SUMMARY OF FOUR DAY SOAKED C.B.R.TEST RESULT

BOREHOLE NUMBER		103
DEPTH (m)		0.30 - 0.80
Surcharge (kg)		4.5
Maximum Dry Density (t/m^3)		1.50 STD
Optimum Moisture Content (%)		27.7
Moulded Dry Density (t/m^3)		1.47
Sample Density Ratio (%)		98
Sample Moisture Ratio (%)		100
Moisture Contents		
Insitu (%)		28.2
Moulded (%)		27.7
After soaking and		
After Test, Top 30mm(%)		32.8
Remaining Depth (%)		30.6
Material Retained on 19mm Sieve (%)		0
Swell (%)		0.0
C.B.R. value:	@5.0mm penetration	3.0

NOTES:

- Refer to appropriate Borehole logs for soil descriptions
- Test Methods :
  - (a) Soaked C.B.R. : AS 1289 6.1.1
  - (b) Standard Compaction : AS 1289 5.1.1
  - (c) Moisture Content : AS 1289 2.1.1

NATA
NATA Accredited Laboratory
Number:1327

Approved Signatory/
Authorised Signature
(A.Tatikonda)

Date: 16/10/07

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Macquarie Park, NSW 2113
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North Ryde, Bc 1670
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SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 21570V
Table C: Page 1 of 2

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S\,(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
101	6.80-6.83	0.8	16
	7.76-7.79	1.0	20
	8.37-8.40	1.9	38
	9.30-9.34	1.6	32
	10.36-10.41	0.8	16
	11.25-11.28	1.1	22
	12.41-12.45	1.2	24
	13.38-13.43	1.0	20
	13.85-13.89	1.0	20
102	7.73-7.77	1.8	36
	8.37-8.40	1.5	30
	9.69-9.73	1.4	28
103	7.21-7.24	0.7	14
	7.85-7.88	0.9	18
	8.15-8.18	0.8	16
	9.62-9.65	1.0	20
104	7.82-7.85	0.6	12
	8.31-8.34	0.7	14
	9.29-9.32	0.7	14
	10.47-10.51	0.7	14
	11.51-11.55	0.9	18
	12.32-12.36	0.8	16
	13.50-13.55	0.8	16
105	8.91-8.95	0.3	6
	9.91-9.95	0.4	8
	10.19-10.23	0.7	14

NOTES:SEE PAGE 2 OF 2

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SOIL TEST SERVICES
ABN 43 002 145 173

Ref No: 21570V
Table C: Page 2 of 2

TABLE C
SUMMARY OF POINT LOAD STRENGTH INDEX TEST RESULTS

BOREHOLE NUMBER	DEPTH m	$I_{S(50)}$ MPa	ESTIMATED UNCONFINED COMPRESSIVE STRENGTH (MPa)
105	10.68-10.71	0.8	16
	11.32-11.35	1.0	20
	12.51-12.55	0.7	14
	13.70-13.74	0.6	12
	14.21-14.25	0.7	14

NOTES:

1. In the above table testing was completed in the Axial direction.
2. The above strength tests were completed at the 'as received' moisture content.
3. Test Method: RTA T223.
4. The Estimated Unconfined Compressive Strength was calculated from the point load Strength Index by the following approximate relationship and rounded off to the nearest whole number :
   U.C.S. = 20 $I_{S(50)}$

LabPoint
LabPoint Pty Ltd
ABN 82 096 903 749
Phone: (02) 9624 5588
Fax: (02) 9624 2266
E-Mail: labpoint@bigpond.net.au
Unit 31, 35 Foundry Road
Seven Hills NSW 2147
P.O.Box 177
Kings Langley 2147

Report No: NAA07-2512

Date Received: 10/10/2007

Order No: C.O.C dated 09/10/07

Attention: Mr. Ashwin Tatikonda

Soil Test Services Pty Ltd
115 Wicks Road
Macquarie Park NSW 2113

Type of Samples: Two soil samples - project 21570V. Analysed 'as received'

Tests	BH 104 3.0-3.45m	BH 105 4.5-4.95m	Methods
pH	5.1	NA	AS 1289 4.3.1 - 1997
Sulphate	NA	<50	AS 1289 D2.1 1997 & APHA 4500 SO_4^{2-} - E

Note: Units: mg/kg dryweight for soils except pH. Analysed "as received".

Samples will be disposed of seven days after issue of this report unless otherwise notified.

The above soil samples have been prepared by customer as follows:
(a) Oven dried at 50 C
(b) Sieved over 2.36 mm sieve
NA means not analysed

Ramabhat S.

Dr Rama Bhat
Manager Environmental Services
Date Issued: 16/10/2007
naa072512.soi 21570V

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

This document is issued in accordance with NATA's accreditation requirements.

Accreditation for compliance with ISO/IEC 17025
Laboratory No. 11111

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
101
1/3

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 24-9-07
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 71.4m
Datum: AHD
Logged/Checked by: G.F./

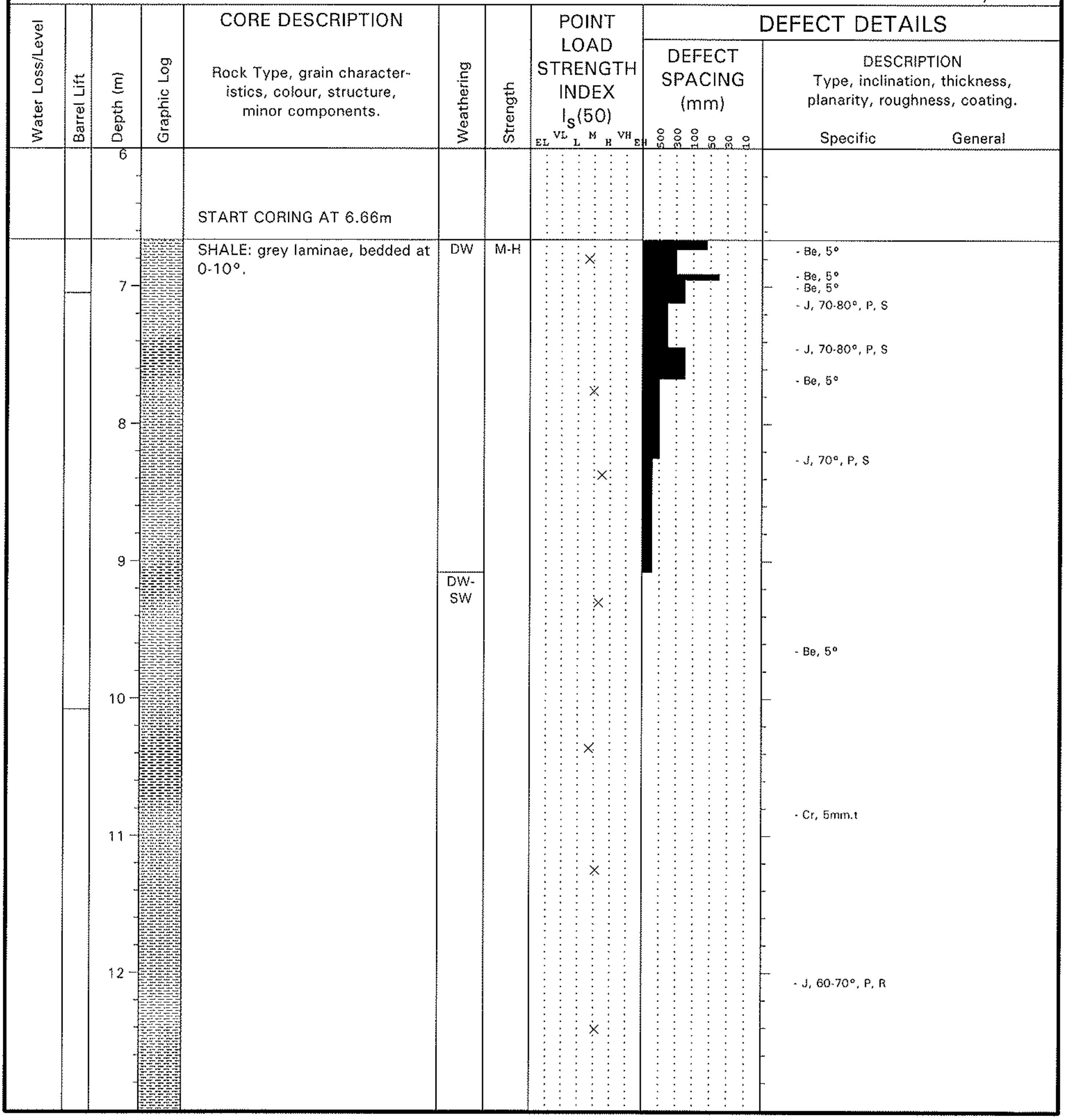
Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			FILL: Silty sand, fine to medium grained, grey brown, with fine to medium grained igneous gravel.	D M			APPEARS MODERATELY COMPACTED
				■	N = 7 2,3,4			CL-CH	FILL: Silty sandy gravel, fine to medium grained, dark grey and black, igneous gravel.	MC≥PL	St	- 180 240 210	POSSIBLY FILL
▼ AFTER 15 DAYS						1			SILTY CLAY: medium to high plasticity, brown, with fine to medium grained ironstone gravel.				
									SILTY CLAY: medium to high plasticity, light grey mottled red brown and orange brown, with fine to medium grained ironstone gravel.	MC≥PL	H		
▼ 1 HR AFTER PUMPING				■	N = 19 7,9,10	2						420 440 400	
				■	N = 20 6,9,11	3					VSt-H	370 360 380	
						4		CL	SILTY CLAY: medium plasticity, light grey, with fine to medium grained ironstone gravel.				POSSIBLY XW SHALE
				■	N > 23 19,23/ 100mm REFUSAL							380 420	
						5		-	SHALE: grey and brown.	DW	L	-	LOW 'TC' BIT RESISTANCE WITH MODERATE BANDS
				■		6							
				■					SHALE: grey.		M-H		MODERATE TO HIGH RESISTANCE
						7			REFER TO CORED BOREHOLE LOG				

CORED BOREHOLE LOG

Borehole No.
101
2/3

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 24-9-07
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 71.4m
Datum: AHD
Logged/Checked by: G.F.

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB NO: 21570V, BH101, START CORING FROM 6.66m
END AT 14.16m
6
7
8
9
10
11
12
13
14

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
101
3/3

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 24-9-07
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 71.4m
Datum: AHD
Logged/Checked by: G.F./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		14	▨	SHALE: grey, laminae, bedded at 0-10°.	DW-SW	M-H	× ×		- J, 80-90°, P, S	
		15 16 17 18 19		END OF BOREHOLE AT 14.16m					14.16m PVC STANDPIPE INSTALLED, BOTTOM 8m SLOTTED, PUMPED TO 5.45m	

BOREHOLE LOG

Borehole No.
102
1/3

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 27-9-07
Method: HAND AUGER/ MELVELLE
R.L. Surface: ≈ 69.6m
Datum: AHD
Logged/Checked by: G.F./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION					REFER TO DCP TEST RESULTS	0	▩		FILL: Silty sand, fine to medium grained, brown, with a trace of clay, fine to medium grained gravel and metal wire.	D			APPEARS MODERATELY COMPACTED
				■		1	▨	CH	SILTY CLAY: high plasticity, brown, with fine to medium grained ironstone gravel.	MC>PL	(F-St)	-	
				■ ■		2 3 4 5 6 7	▨	CL-CH	SILTY CLAY: medium to high plasticity, light brown, red brown and light grey, with fine to medium grained ironstone gravel.		(VSt)		HAND AUGER REFUSAL COMMENCED WASHBORING FROM 1.95m

BOREHOLE LOG

Borehole No.
102
2/3

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 27-9-07
Method: HAND AUGER/ MELVELLE
Logged/Checked by: G.F./
R.L. Surface: ≈ 69.6m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
									REFER TO CORED BOREHOLE LOG				END WASHBORING
						8							
						9							
						10							
						11							
						12							
						13							
						14							


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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB NO: 21570V, BH102, START CORING FROM 7.15m
END AT 10.03m
7
8
9
10

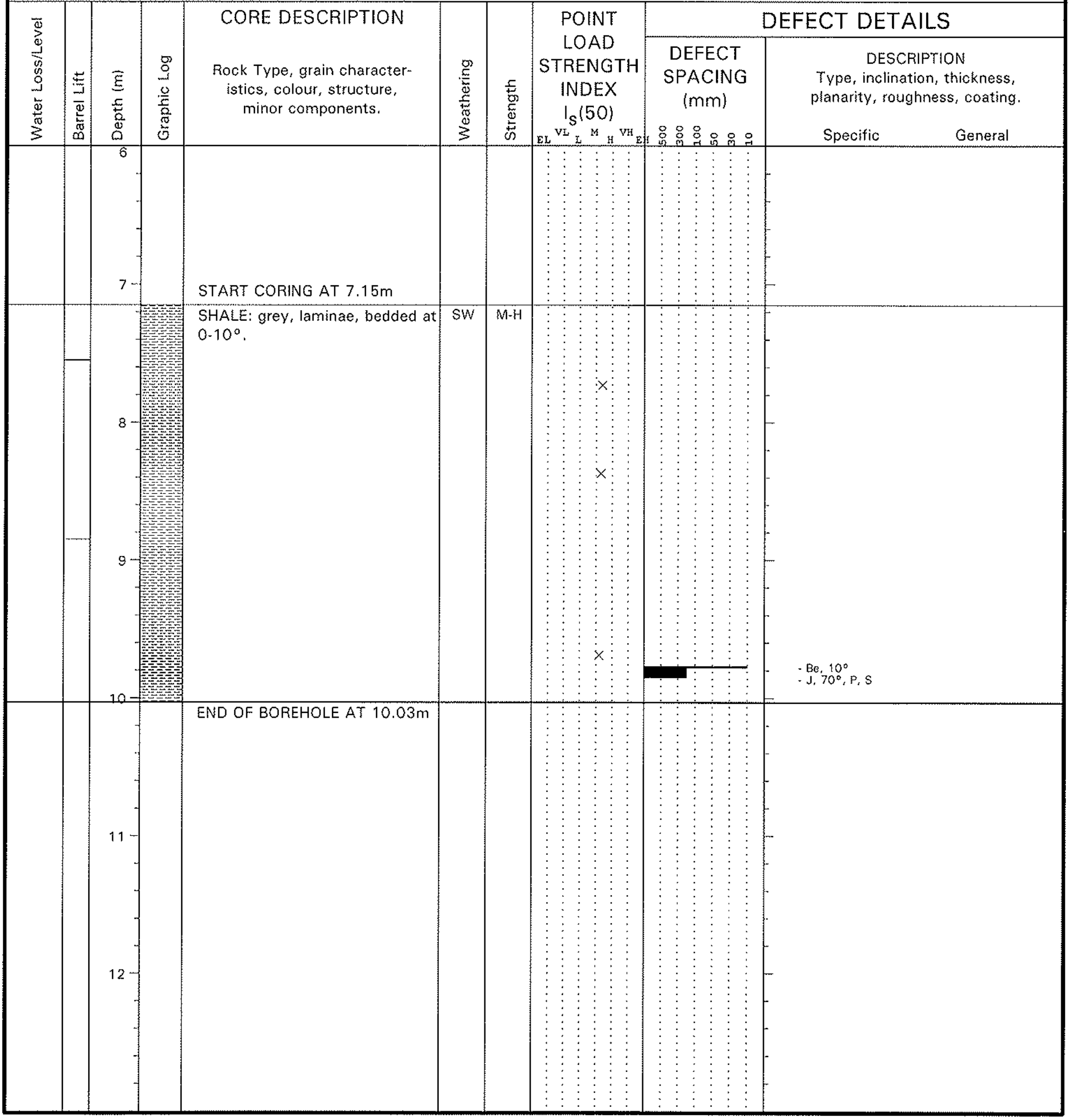
CORED BOREHOLE LOG

Borehole No.
102
3/3

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Core Size: NMLC
R.L. Surface: ≈ 69.6m
Date: 27-9-07
Inclination: VERTICAL
Datum: AHD
Drill Type: MELVELLE
Bearing: -
Logged/Checked by: G.F./ W

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT SPACING (mm)	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
				START CORING AT 7.15m						
				SHALE: grey, laminae, bedded at 0-10°.	SW	M-H			- Be, 10° - J, 70°, P, S	
				END OF BOREHOLE AT 10.03m						



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BOREHOLE LOG

Borehole No.
103
1/2

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 28-9-07
Method: SPIRAL AUGER/ MELVELLE
R.L. Surface: ≈ 70.5m
Datum: AHD
Logged/Checked by: J.M./

Groundwater Record	SAMPLES				Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
	ES	U50	DB	DS									
DRY ON COMPLETION			■		REFER TO DCP TEST RESULTS	0		CH	CONCRETE: 55mm.t SILTY CLAY: high plasticity, light brown, with ironstone gravel.	MC>PL	(F-St)	-	
													COMMENCED WASHBORING AT 0.65m
											(VSt)		
						1							
▼ AFTER CORING													
▼ AFTER 11 DAYS													
						2							
						3							
						4							
						5							
						6							
									REFER TO CORED BOREHOLE LOG				END WASHBORING
						7							

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JOB No: 21570V BH 103 START CORING AT 6·70m
6·70m
C.L 0·09m
7
8
CORE LOSS 0·21m
9

CORED BOREHOLE LOG

Borehole No.
103
2/2

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Core Size: NMLC
R.L. Surface: ≈ 70.5m
Date: 28-9-07
Inclination: VERTICAL
Datum: AHD
Drill Type: MELVELLE
Bearing: -
Logged/Checked by: J.M./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		6		START CORING AT 6.70m						
				SHALE: grey, with light grey laminae.	DW	M-H				
				CORE LOSS 0.09m						
		7 8		SHALE: grey, with light grey laminae.	SW	M-H	× × ×		- J, 45°, P, R	
				CORE LOSS 0.21m						
				SHALE: grey, with light grey laminae.	SW	M-H				
				CORE LOSS 0.19m						
		9		SHALE: grey, with light grey laminae.	SW	M-H	×		- J, 45°, P, S - Be, 15mm.t	
		10 11 12		END OF BOREHOLE AT 10.00m					10mm PVC STANDPIPE INSTALLED, BOTTOM 3m SLOTTED, BAILED TO 2.75m	

BOREHOLE LOG

Borehole No.
104
1/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 26-9-07
Method: SPIRAL AUGER
JK250
R.L. Surface: ≈ 70.0m
Datum: AHD
Logged/Checked by: G.F./

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGER-ING						0			CONCRETE: 250mm.t				
								-	FILL: Silty sandy gravel, fine to medium grained, brown, igneous gravel.	M	-	-	
				■	N = 7 2,3,4			CL-CH	SILTY CLAY: medium to high plasticity, light brown, with fine to medium grained ironstone gravel.	MC>PL	St	- 200 210 190	
						1							
									SILTY CLAY: medium to high plasticity, light grey mottled red brown and orange brown, with fine to medium grained ironstone gravel.		VSt -H		
				■	N = 18 7,9,9							430 400 410	
						2							
									as above, but medium plasticity.		H		
				■	N = 22 9,9,13	3						410 530 540	
						4							POSSIBLY XW SHALE WITH IRON INDURATED BANDS
				■	N > 38 13,16, 22/100mm REFUSAL							>600 >600 >600	
						5							
								-	SHALE: light grey, with iron indurated bands.	XW	EL	-	
				■	SPT 15/20mm REFUSAL	6							
									SHALE: dark grey, with iron indurated bands.	DW	L-M		LOW TO MODERATE RESISTANCE
						7							

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

BOREHOLE LOG

Borehole No.
104
2/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 26-9-07
Method: SPIRAL AUGER JK250
R.L. Surface: ≈ 70.0m
Datum: AHD
Logged/Checked by: G.F./

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
				■					SHALE: dark grey, with iron indurated bands.	DW	L-M		
									REFER TO CORED BOREHOLE LOG				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

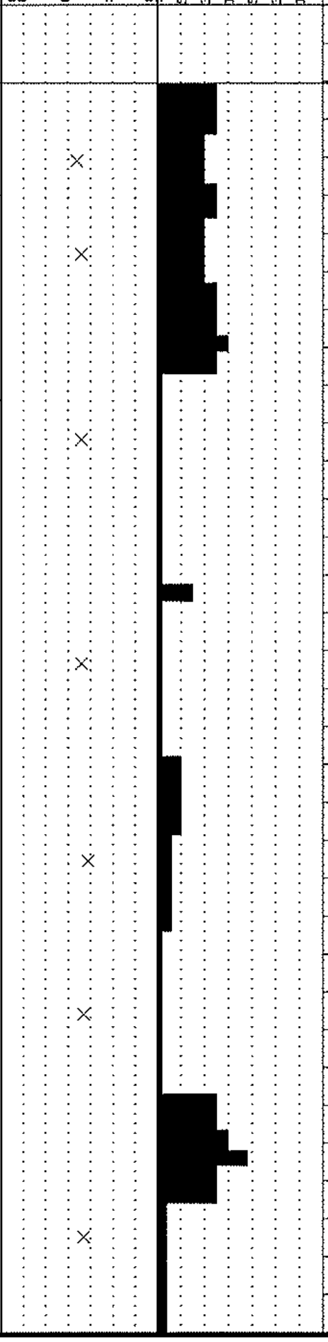
CORED BOREHOLE LOG

Borehole No.
104
3/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 26-9-07
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 70.0m
Datum: AHD
Logged/Checked by: G.F./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		7		START CORING AT 7.41m						
				SHALE: grey, with iron indurated bands, laminae, bedded at 0-10°.	DW	L-M	×		- FRACTURED ZONE, 30mm.t - J, 40-50°, P, S	
		8				M	×		- J, 20-30°, P, S - XWS, 5mm.t - J, 30-40°, P, S - J, 20°, P, S - J, 60-80°, P, R	
		9		SHALE: grey laminae, bedded at 0-15°	DW-SW	M	×		- J, 40-50°, P, S	
		10					×		- J, 60-70°, P, S - J, 20-30°, P, S	
		11					×		- J, 20-30°, P, S - FRACTURED ZONE, 5mm.t - J, 50-60°, P, R	
		12					×		- XWS, 10mm.t	
		13					×		- XWS, 30mm.t - J, 20°, P,S - J, 30-40°, P, S - J, 30°, P, S	

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JOB NO: 21570V, BH104, START CORING FROM 7.41m
END AT 14.13m
7
8
9
10
11
12
13
14

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CORED BOREHOLE LOG

Borehole No.
104
4/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Core Size: NMLC
R.L. Surface: ≈ 70.0m
Date: 26-9-07
Inclination: VERTICAL
Datum: AHD
Drill Type: JK250
Bearing: -
Logged/Checked by: G.F./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$ EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
				SHALE: grey laminae, bedded at 0-15°						
				END OF BOREHOLE AT 14.13m						
		15								
		16								
		17								
		18								
		19								
		20								

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BOREHOLE LOG

Borehole No.
105
1/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 25-9-07
Method: SPIRAL AUGER
JK250
Logged/Checked by: G.F./
R.L. Surface: ≈ 74.3m
Datum: AHD

Groundwater Record	SAMPLES ES	SAMPLES U50	SAMPLES DB	SAMPLES DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
DRY ON COMPLETION OF AUGERING						0			CONCRETE: 90mm.t				
								-	FILL: Silty clay, medium to high plasticity, light brown, red brown and grey, with fine to medium grained ironstone and igneous gravel, with fine to medium grained sand.	MC>PL	-	-	APPEARS POORLY COMPACTED
				■	N = 4 3,2,2								
						1			FILL: Silty clay, medium to high plasticity, grey and brown, with fine to medium grained ironstone and igneous gravel, brick fragments and fine to medium grained sand.				APPEARS MODERATELY TO WELL COMPACTED
				■	N = 9 5,4,5								
						2							
				■	N = 2 0,1,1	3			FILL: Silty sand, fine to medium grained, grey, with fine to medium grained igneous and ironstone gravel and clay fines.	M			APPEARS POORLY COMPACTED
						4		CL-CH	SILTY CLAY: medium to high plasticity, light grey and orange brown, with fine to medium grained ironstone gravel.	MC>PL	(VSt)	-	
				■	N = 19 4,8,11						VSt	390 380 400	
						5							
								-	SHALE: light grey, with iron indurated bands.	XW	EL	-	
				■	N > 21 18,21, 15/50mm REFUSAL	6						>600 >600 >600	
						7				DW	L-M		LOW TO MOD.

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BOREHOLE LOG

Borehole No.
105
2/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 25-9-07
Method: SPIRAL AUGER JK250
Logged/Checked by: G.F./
R.L. Surface: ≈ 74.3m
Datum: AHD

Groundwater Record	SAMPLES ES	U50	DB	DS	Field Tests	Depth (m)	Graphic Log	Unified Classification	DESCRIPTION	Moisture Condition/ Weathering	Strength/ Rel. Density	Hand Penetrometer Readings (kPa.)	Remarks
▼ AFTER 14 DAYS				■			▨		SHALE: grey.	DW	L-M		RESISTANCE
									REFER TO CORED BOREHOLE LOG				
						8							
						9							
						10							
						11							
						12							
						13							
						14							

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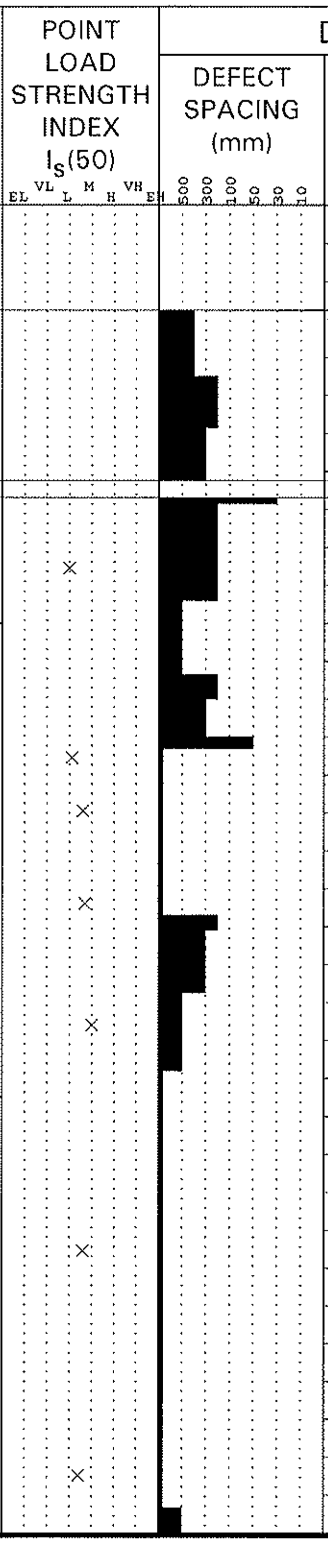
Borehole No.
105
3/4

CORED BOREHOLE LOG

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Core Size: NMLC
R.L. Surface: ≈ 74.3m
Date: 25-9-07
Inclination: VERTICAL
Datum: AHD
Drill Type: JK250
Bearing: -
Logged/Checked by: G.F./



Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	DEFECT DETAILS DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
		7		START CORING AT 7.55m				
▼ 18 HRS AFTER PUMPING		8		SHALE: grey, with iron indurated bands.	DW	L	- J, 50-85°, Un, R - FRACTURED ZONE, 130mm.t	
				CORE LOSS 0.09m				
		9		SHALE: grey laminae, bedded at 0-10°.	DW-SW	L-M	- J, 50°, P, S - J, 20°, P, S - Cr, 5mm.t	
						M	- XWS, 5mm.t - XWS, 15mm.t - J, 30-40°, P, S - FRACTURED ZONE, 10mm.t	
		10						
		11					- J, 5-10°, P, S - XWS, 10mm.t - XWS, 20mm.t - J, 60-80°, Un, S	
					SW	M-H		
		12						
		13					- J, 30°, P, S	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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS
JOB. NO: 21570V, BH105, START CORING FROM 7.55m
CORE LOSS 0.09m
END AT 14.30m
7
8
9
10
11
12
13
14

Jeffery and Katauskas Pty Ltd
CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

CORED BOREHOLE LOG

Borehole No.
105
4/4

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 25-9-07
Drill Type: JK250
Core Size: NMLC
Inclination: VERTICAL
Bearing: -
R.L. Surface: ≈ 74.3m
Datum: AHD
Logged/Checked by: G.F./

Water Loss/Level	Barrel Lift	Depth (m)	Graphic Log	CORE DESCRIPTION Rock Type, grain characteristics, colour, structure, minor components.	Weathering	Strength	POINT LOAD STRENGTH INDEX $I_s(50)$	DEFECT DETAILS		
							EL VL L M H VH EH	DEFECT SPACING (mm) 500 300 100 50 30 10	DESCRIPTION Type, inclination, thickness, planarity, roughness, coating. Specific	General
			▨	SHALE: grey laminae, bedded at 0-15°			×	■		
				END OF BOREHOLE AT 14.30m					14.30m PVC STANDPIPE INSTALLED, BOTTOM 8m SLOTTED, PUMPED TO 13.0m	
		15								
		16								
		17								
		18								
		19								
		20								

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CONSULTING GEOTECHNICAL AND ENVIRONMENTAL ENGINEERS

DYNAMIC CONE PENETRATION TEST RESULTS

Client: EASTWOOD CENTRE DEVELOPMENTS PTY LTD
Project: REDEVELOPMENT OF SHOPPING CENTRE
Location: CNR. RUTLEDGE AND TRELAWNEY STREETS, EASTWOOD, NSW

Job No. 21570V
Date: 27-9--07
Tested By: G.F./B.Z.

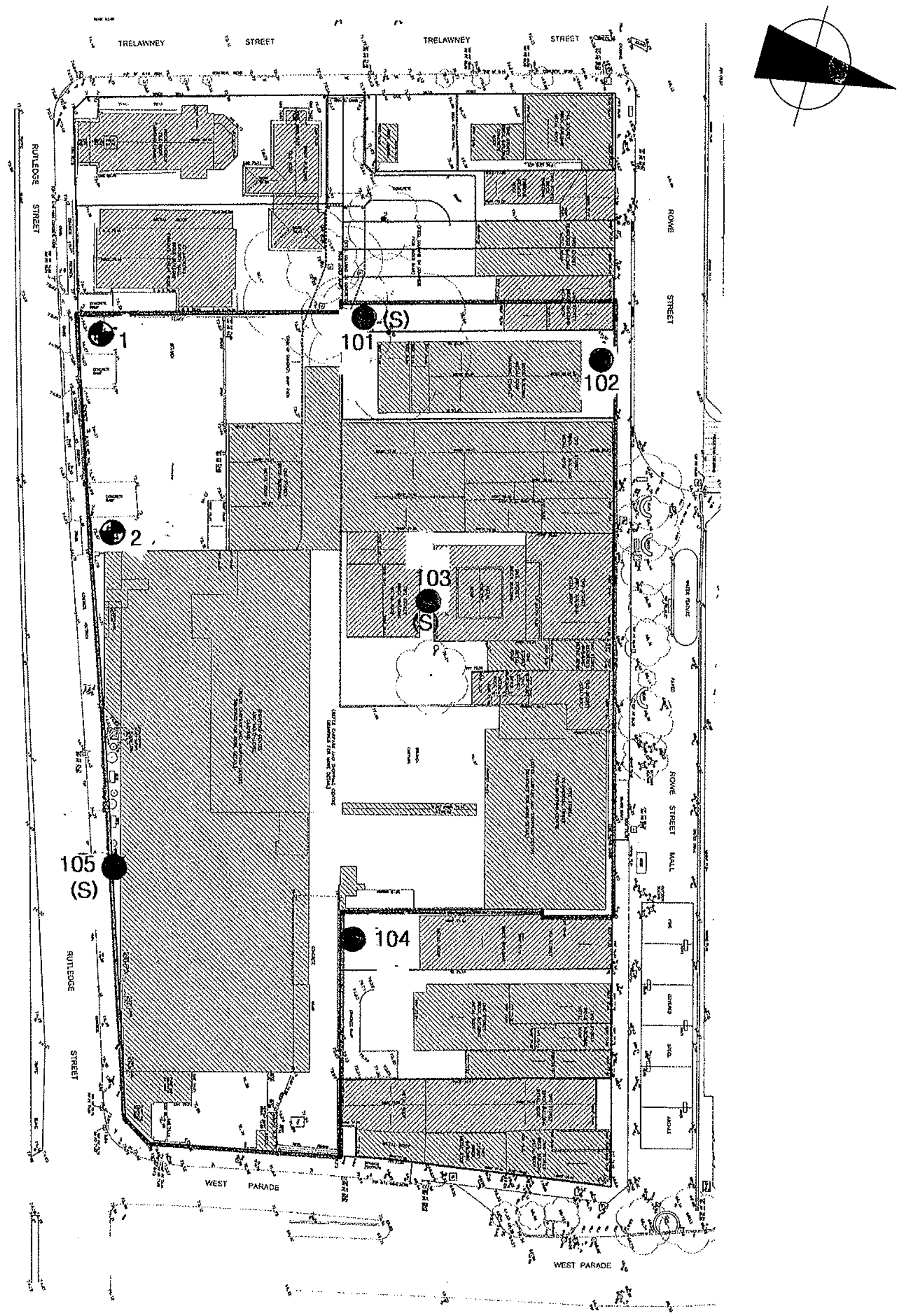
Hammer Weight & Drop: 9kg/510mm
Rod Diameter: 16mm
Point Diameter: 20mm

Number of Blows per 100mm Penetration

Test Location	RL ~69.6m	RL ~70.5m					
Depth (mm)	**101**	**103**					
0 - 100	5	VOID					
100 - 200	8	1					
200 - 300	6	↓					
300 - 400	5	4					
400 - 500	3	1					
500 - 600	4	2					
600 - 700	2	2					
700 - 800	2	3					
800 - 900	2	4					
900 - 1000	2	5					
1000 - 1100	3	5					
1100 - 1200	3	8					
1200 - 1300	4	8					
1300 - 1400	4	9					
1400 - 1500	4	11					
1500 - 1600	6	12					
1600 - 1700	5	14					
1700 - 1800	6	13					
1800 - 1900	5	14					
1900 - 2000	6	25/60mm					
2000 - 2100	6	REFUSAL					
2100 - 2200	6						
2200 - 2300	9						
2300 - 2400	8						
2400 - 2500	15						
2500 - 2600	19						
2600 - 2700	30/90mm						
2700 - 2800	REFUSAL						
2800 - 2900							
2900 - 3000							

Remarks:
1. The procedure used for this test is similar to that described in AS1289.6.3.2-1997, Method 6.3.2.
2. Usually 8 blows per 20mm is taken as refusal

Ref: Scala3.xls April 99



LEGEND

- ● **BOREHOLE**
- ◐ **BOREHOLE (DOUGLAS 2004)**
- (S) **GROUNDWATER MONITORING WELL**

SCALE (m)

0 50

BOREHOLE LOCATION PLAN

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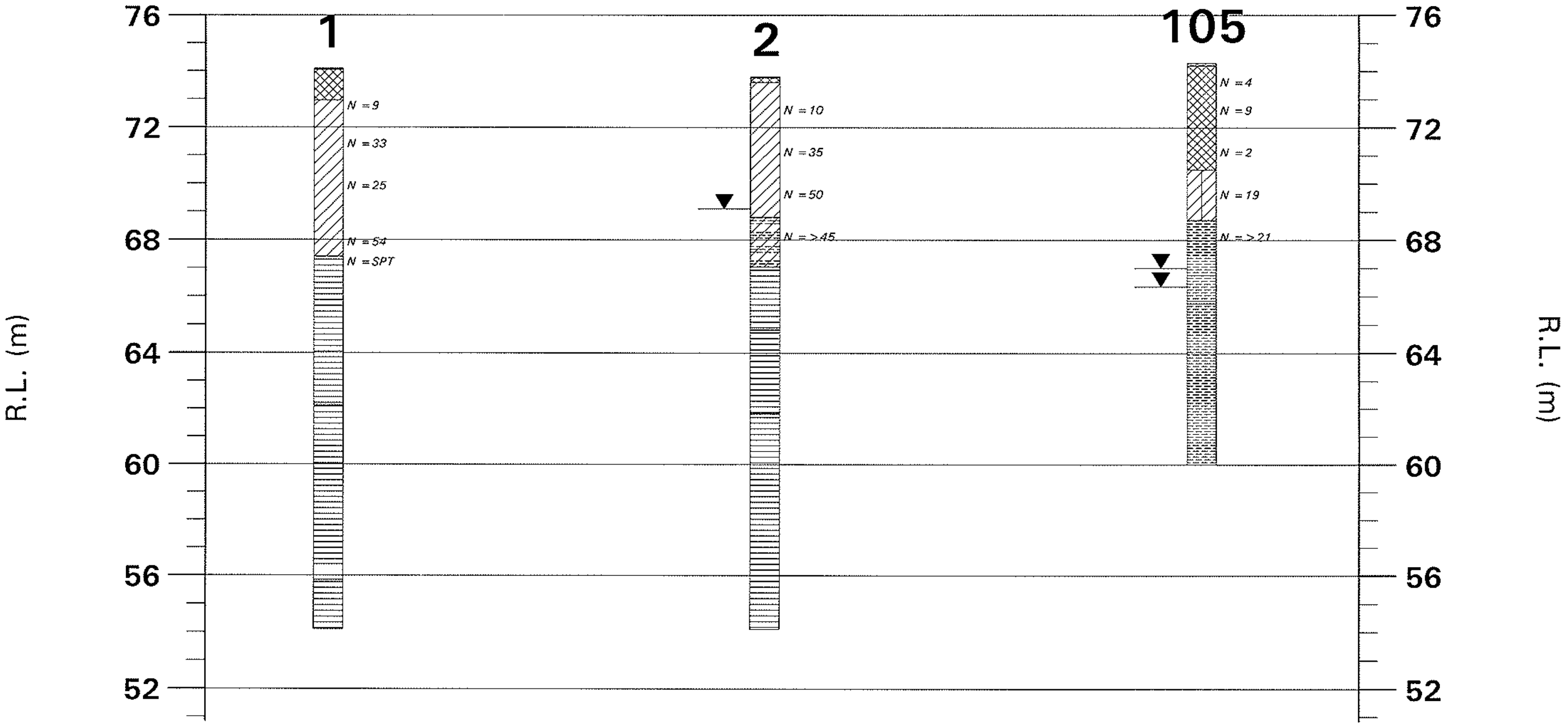
Report No. 21570V **Figure No. 1**

GRAPHICAL BOREHOLE SUMMARY



■ Asphaltic/ Bituminous Paving or Coal

▩ Fill

▨ Clay

☰ Siltstone, Mudstone, Claystone

☐ Core Loss/ Empty

▨ Shaly Clay

▦ Shale

▒ Concrete

▥ Silty Clay

▼ Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

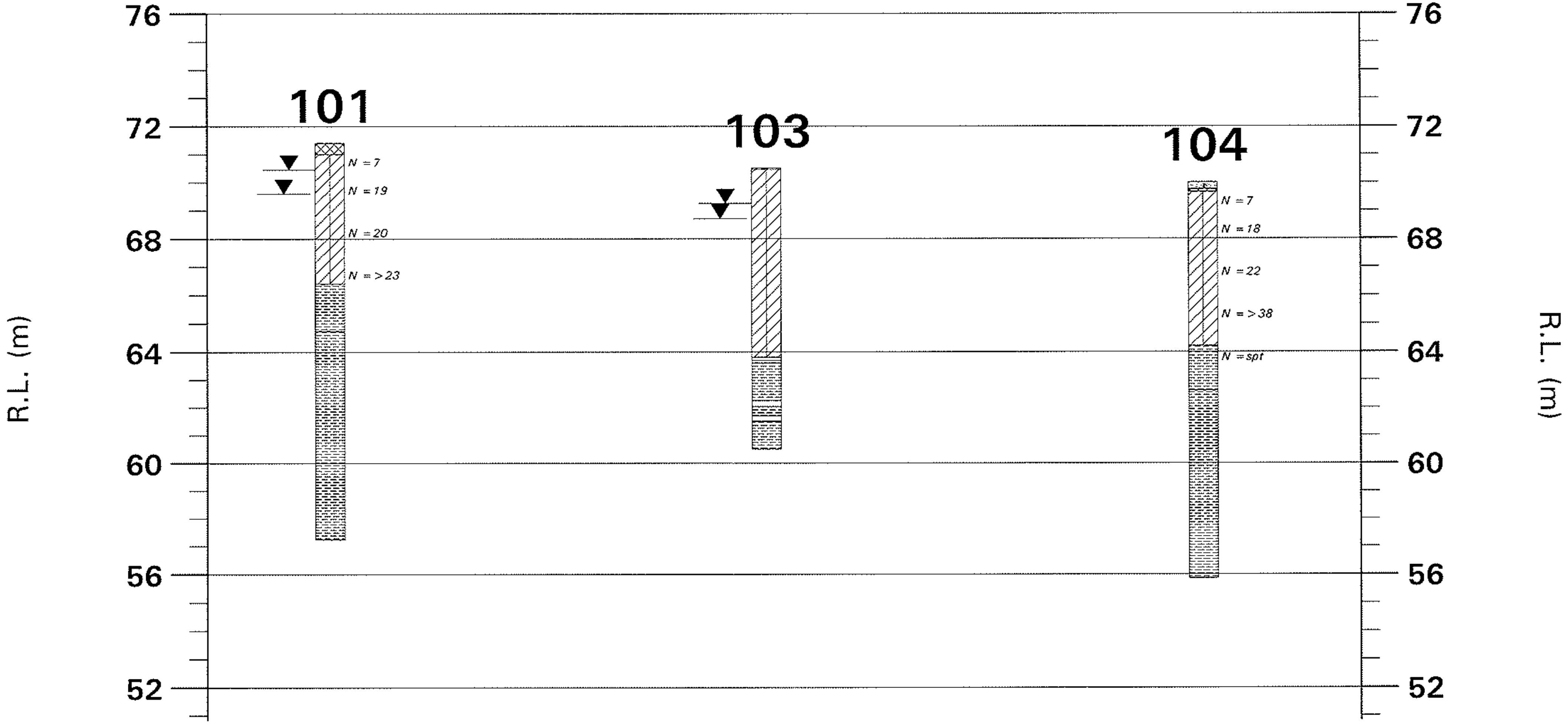
Scale: 1 : 200 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 21570V

Figure No.: 2A

GRAPHICAL BOREHOLE SUMMARY



▤ Shale
▩ Fill
▨ Silty Clay
░ Concrete
☐ Core Loss/ Empty
▼ Observed water level

N	SPT "N" VALUE
Nc	SOLID CONE BLOW COUNTS PER 150mm

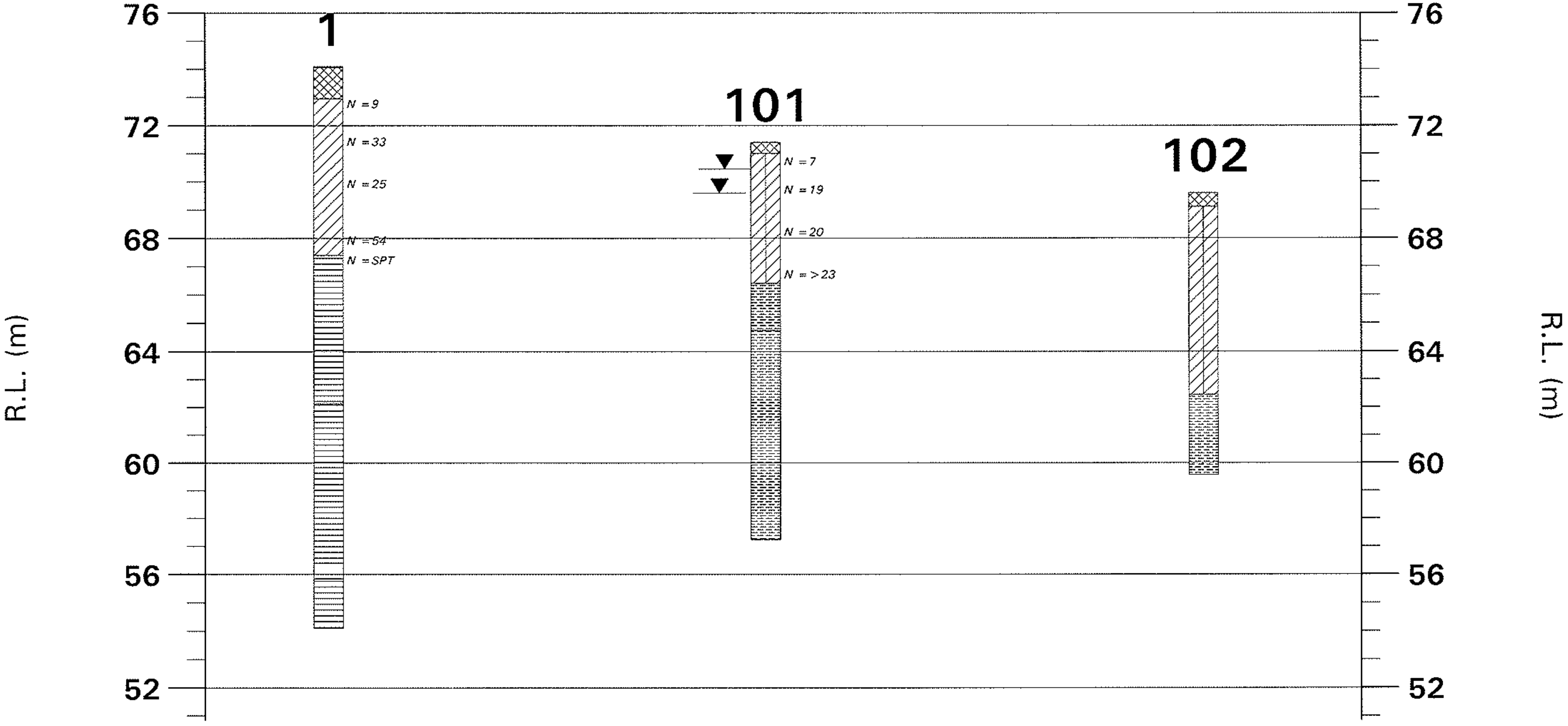
NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 21570V **Figure No.:** 2B

GRAPHICAL BOREHOLE SUMMARY



■ Asphaltic/ Bituminous Paving or Coal

▩ Fill

▨ Clay

☰ Siltstone, Mudstone, Claystone

☐ Core Loss/ Empty

▨ Silty Clay

▤ Shale

▼ Observed water level

N SPT "N" VALUE

Nc SOLID CONE BLOW COUNTS PER 150mm

NOTE: REFER TO BOREHOLE LOGS

Scale: 1 : 200 (vert) ; NTS (horiz)

Jeffery and Katauskas Pty Ltd

Job No.: 21570V

Figure No.: 2C

APPENDIX A

TEST BORE REPORT

BORE No: 1

CLIENT: Bernard Chan Nominees Pty Ltd
PROJECT: Proposed Redevelopment
LOCATION: Eastwood Centre, Rutledge St, Eastwood

PROJECT No: 36766
SURFACE LEVEL: 74.1m AHD
DIP OF HOLE: 90°

DATE: 12-17/2/04
SHEET 1 OF 2
AZIMUTH: --

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALT					
0.15	ROADBASE GRAVEL					
	FILLING - brown clay filling, with crushed brick, roadbase gravel, ironstone gravel and a trace of slag		A A			
1.15	CLAY - stiff, mottled orange and yellow brown clay, with some ironstone grave. Damp		S A			4,3,6 N=9
1.8	CLAY - hard, light grey and orange brown clay, with ironstone gravel. Damp		A			
2.5	CLAY - hard, light grey mottled red brown clay, with ironstone gravel bands. Damp		S S			7,13,20 N = 33 8,12,13 N = 25
4.5	CLAY - very stiff, grey mottled orange brown clay, with ironstone gravel bands and a trace of decomposed roots	Note: Unless otherwise stated, rock is fractured along planar bedding planes dipping 0°- 5°	S			13,21,33 N = 54
6.7	SILTSTONE - extremely low and low strength, extremely and highly weathered, light grey, orange brown and dark grey siltstone		S			25/70mm refusal
7.19	SILTSTONE - medium strength, fresh stained, fractured, dark grey/black siltstone with 10% light grey fine grained sandstone laminae	7.19-8.0m: bedding partings ironstained 7.23m: J35° stepped to bedding ironstained, healed 7.41m: J45° curvilinear to 90° ironstained, healed	C	100	0	PL(A) = 0.6MPa
8.0	SILTSTONE - medium and high strength, fractured then slightly fractured, dark grey black siltstone, with 20% light grey fine grained sandstone laminae	8.1m: B, 2mm clay 8.23m: B, 2mm clay 8.38m: B, 1mm clay	C C	100 98	0 98	PL(A) = 1.3MPa PL(A) = 0.8MPa PL(A) = 0.7MPa

RIG: SCOUT **DRILLER:** L COOPER **LOGGED:** JARDINE **CASING:** TO 2.5m

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 2.5m; ROTARY TO 7.1m; NMLC-CORING TO 20.0m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: DEM
Date: 14/4

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TEST BORE REPORT

BORE No: 1

CLIENT: Bernard Chan Nominees Pty Ltd
PROJECT No: 36766
DATE: 12-17/2/04

PROJECT: Proposed Redevelopment
SURFACE LEVEL: 74.1m AHD
SHEET 2 **OF** 2

LOCATION: Eastwood Centre, Rutledge St, Eastwood
DIP OF HOLE: 90°
AZIMUTH: --

Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
	SILTSTONE - medium and high strength, fractured then slightly fractured, dark grey black siltstone, with 20% light grey fine grained sandstone laminae *(continued)*	11.34m: J35° rough, stepped. Minor pyrite mineralisation	C	98	98	
11.95		11.95m: CORE LOSS: 50mm	C	100	100	PL(A) = 1.4MPa
		16.68m: J85° planar, healed, incipient	C	90	90	PL(A) = 0.6MPa
17.7		17.7m: CORE LOSS: 560mm				
			C	87	87	PL(A) = 0.6MPa PL(A) = 0.8MPa
20.0	TEST BORE DISCONTINUED AT 20.0m					

RIG: SCOUT **DRILLER:** L COOPER **LOGGED:** JARDINE **CASING:** TO 2.5m

TYPE OF BORING: SPIRAL FLIGHT AUGER TO 2.5m; ROTARY TO 7.1m; NMLC-CORING TO 20.0m

WATER OBSERVATIONS: NO FREE GROUNDWATER OBSERVED WHILST AUGERING

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U,	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: DEM
Date: 14/4

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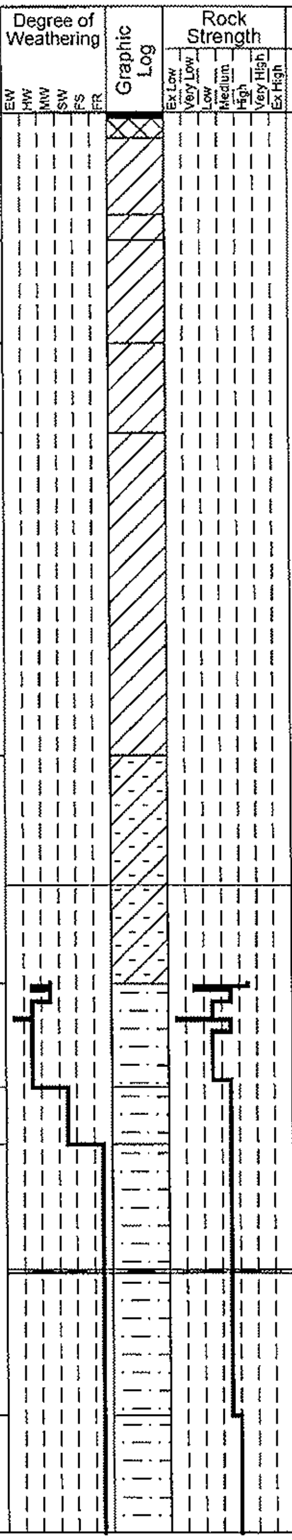
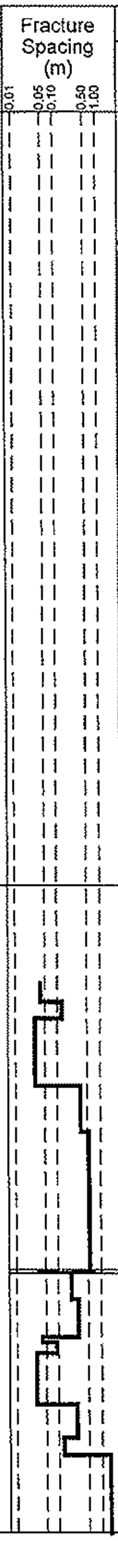
TEST BORE REPORT

BORE No: 2

CLIENT: Bernard Chan Nominees Pty Ltd
PROJECT No: 36766
DATE:
PROJECT: Proposed Redevelopment
SURFACE LEVEL: 73.8m AHD
SHEET 1 OF 2
LOCATION: Eastwood Centre, Rutledge St, Eastwood
DIP OF HOLE: 90°
AZIMUTH: --





Depth (m)	Description of Strata	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
0.05	ASPHALT		A			
0.2	FILLING - brown clay filling, with some gravel		A			
	CLAY - orange and red brown clay. Damp					
0.8	CLAY - brown clay, slightly sandy. Damp		A			
1.0	CLAY - stiff, mottled yellow and orange brown clay with some ironstone gravel. Damp		S A			3,4,6 N=10
1.8	CLAY - light grey and orange brown clay, with some ironstone gravel. Damp		A			
2.5	CLAY - hard, light grey mottled red brown clay, with ironstone gravel. Damp		S S			10,15,20 N = 35 14,20,30 N = 50
5.0	SHALY CLAY - hard, light grey shaly clay, with some ironstone gravel and a trace of decomposed roots. Damp	Note: Unless otherwise stated, rock is fractured along planar bedding planes dipping 0°- 5°	S			14,20,25/100mm refusal
6.76	SILTSTONE - low strength, moderately and highly weathered, fractured, orange brown, light and dark grey siltstone. Some extremely low, very low, medium and high strength bands	6.9m: J85°- 90° rough planar, ironstained	C	100	9	PL(A) = 0.2MPa
7.56	SILTSTONE - medium strength, slightly weathered, slightly fractured, dark grey/black and orange brown siltstone	7.9m: J90° rough planar, ironstained				PL(A) = 0.5MPa
8.0	SILTSTONE - medium strength, fresh, slightly fractured and fractured, dark grey/black siltstone. Some clay seams	8.43m: J50° rough stepped to bedding, ironstained	C	98	98	PL(A) = 0.3MPa
8.97		8.97m: CORE LOSS: 30mm 9.2m: J35° rough planar 9.49 & 9.53m: J45° rough planar, calcite mineralisation	C	99	82	
10.1	SILTSTONE - medium to high strength, fresh, slightly fractured, dark grey/black siltstone, with 10% fine grained sandstone laminae	10.27m: J90° rough planar, partially healed				PL(A) = 1.1MPa

RIG: SCOUT **DRILLER:** L COOPER **LOGGED:** JARDINE **CASING:** TO 2.5m

TYPE OF BORING: SPPIRAL FLIGHT AUGER TO 2.5m; ROTARY TO 6.0m; NMLC-CORING TO 19.70m

WATER OBSERVATIONS: CONSTANT SLOW WATER LOSS FROM 12.0 TO 19.70m

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U_x	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: DEM
Date: 14/4

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TEST BORE REPORT

CLIENT: Bernard Chan Nominees Pty Ltd
PROJECT: Proposed Redevelopment
LOCATION: Eastwood Centre, Rutledge St, Eastwood

PROJECT No: 36766
SURFACE LEVEL: 73.8m AHD
DIP OF HOLE: 90°

BORE No: 2
DATE:
SHEET 2 **OF** 2
AZIMUTH: --

Depth (m)	Description of Strata	Degree of Weathering (EW HW MW SW FS FR)	Graphic Log	Rock Strength (Ex Low, Very Low, Low, Medium, High, Very High, Ex High)	Discontinuities (B - Bedding, J - Joint, S - Shear, D - Drill Break)	Fracture Spacing (m) (0.01 0.05 0.10 0.50 1.00)	Sample Type	Core Rec. %	RQD %	Test Results & Comments
11.0	SILTSTONE - as above									PL(A) = 1MPa
11.4	SILTSTONE - medium strength, fresh, slightly fractured, dark grey black siltstone with 10% fine grained sandstone laminae						C	99	82	
11.97					11.97m: CORE LOSS: 30mm		C	100	100	PL(A) = 0.5MPa PL(A) = 0.5MPa
15					17.09m: J35°smooth planar 17.35m: J90° rough planar 17.45m: J90° rough planar 17.62m: J45° smooth planar 17.91m: J75° rough planar 19.21m: J45° smooth, stepped		C	100	100	PL(A) = 0.9MPa PL(A) = 0.5MPa
19.7	TEST BORE DISCONTINUED AT 19.7m									

RIG: SCOUT **DRILLER:** L COOPER **LOGGED:** JARDINE **CASING:** TO 2.5m

TYPE OF BORING: SPPIRAL FLIGHT AUGER TO 2.5m; ROTARY TO 6.0m; NMLC-CORING TO 19.70m

WATER OBSERVATIONS: CONSTANT SLOW WATER LOSS FROM 12.0 TO 19.70m

REMARKS:

SAMPLING & IN SITU TESTING LEGEND

A	Auger sample	PL	Point load strength Is(50) MPa
B	Bulk sample	S	Standard penetration test
C	Core drilling	U,	Tube sample (x mm dia.)
pp	Pocket penetrometer (kPa)	V	Shear vane (kPa)

CHECKED
Initials: DEM
Date: 14/4

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VIBRATION EMISSION DESIGN GOALS

German Standard DIN 4150 – Part 3: 1986 provides guideline levels of vibration velocity for evaluating the effects of vibration in structures. The limits presented in this standard are generally recognised to be conservative.

The DIN 4150 values (maximum levels measured in any direction at the foundation, OR, maximum levels measured in (x) or (y) horizontal directions, in the plane of the uppermost floor), are summarised in Table 1 below.

It should be noted that peak vibration velocities higher than the minimum figures in Table 1 for low frequencies may be quite "safe", depending on the frequency content of the vibration and the actual condition of the structure.

It should also be noted that these levels are "safe limits", up to which no damage due to vibration effects has been observed for the particular class of building. "Damage" is defined by DIN 4150 to include even minor non-structural effects such as superficial cracking in cement render, the enlargement of cracks already present, and the separation of partitions or intermediate walls from load bearing walls. Should damage be observed at vibration levels lower than the "safe limits" then it may be attributed to other causes. DIN 4150 also states that when vibration levels higher than the "safe limits" are present, it does not necessarily follow that damage will occur. Values given are only a broad guide.

Table 1 DIN 4150 – Structural Damage – Safe Limits for Building Vibration

Group	Type of Structure	Peak Vibration Velocity in mm/s			
		At Foundation Level At a Frequency of			Plane of Floor of Uppermost Storey
		Less than 10 Hz	10 Hz to 50 Hz	50 Hz to 100 Hz	All Frequencies
1	Buildings used for commercial purposes, industrial buildings and buildings of similar design.	20	20 to 40	40 to 50	40
2	Dwellings and buildings of similar design and/or use.	5	5 to 15	15 to 20	15
3	Structures that because of their particular sensitivity to vibration, do not correspond to those listed in Group 1 and 2 and have intrinsic value (eg buildings that are under a preservation order).	3	3 to 8	8 to 10	8

Note: For frequencies above 100 Hz, the higher values in the 50 Hz to 100 Hz column should be used.

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REPORT EXPLANATION NOTES

INTRODUCTION

These notes have been provided to amplify the geotechnical report in regard to classification methods, field procedures and certain matters relating to the Comments and Recommendations section. Not all notes are necessarily relevant to all reports.

The ground is a product of continuing natural and man-made processes and therefore exhibits a variety of characteristics and properties which vary from place to place and can change with time. Geotechnical engineering involves gathering and assimilating limited facts about these characteristics and properties in order to understand or predict the behaviour of the ground on a particular site under certain conditions. This report may contain such facts obtained by inspection, excavation, probing, sampling, testing or other means of investigation. If so, they are directly relevant only to the ground at the place where and time when the investigation was carried out.

DESCRIPTION AND CLASSIFICATION METHODS

The methods of description and classification of soils and rocks used in this report are based on Australian Standard 1726, the SAA Site Investigation Code. In general, descriptions cover the following properties – soil or rock type, colour, structure, strength or density, and inclusions. Identification and classification of soil and rock involves judgement and the Company infers accuracy only to the extent that is common in current geotechnical practice.

Soil types are described according to the predominating particle size and behaviour as set out in the attached Unified Soil Classification Table qualified by the grading of other particles present (eg sandy clay) as set out below:

Soil Classification	Particle Size
Clay	less than 0.002mm
Silt	0.002 to 0.06mm
Sand	0.06 to 2mm
Gravel	2 to 60mm

Non-cohesive soils are classified on the basis of relative density, generally from the results of Standard Penetration Test (SPT) as below:

Relative Density	SPT 'N' Value (blows/300mm)
Very loose	less than 4
Loose	4 – 10
Medium dense	10 – 30
Dense	30 – 50
Very Dense	greater than 50

Cohesive soils are classified on the basis of strength (consistency) either by use of hand penetrometer, laboratory testing or engineering examination. The strength terms are defined as follows.

Classification	Unconfined Compressive Strength kPa
Very Soft	less than 25
Soft	25 – 50
Firm	50 – 100
Stiff	100 – 200
Very Stiff	200 – 400
Hard	Greater than 400
Friable	Strength not attainable – soil crumbles

Rock types are classified by their geological names, together with descriptive terms regarding weathering, strength, defects, etc. Where relevant, further information regarding rock classification is given in the text of the report. In the Sydney Basin, "Shale" is used to describe thinly bedded to laminated siltstone.

SAMPLING

Sampling is carried out during drilling or from other excavations to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on plasticity, grain size, colour, moisture content, minor constituents and, depending upon the degree of disturbance, some information on strength and structure. Bulk samples are similar but of greater volume required for some test procedures.

Undisturbed samples are taken by pushing a thin-walled sample tube, usually 50mm diameter (known as a U50), into the soil and withdrawing it with a sample of the soil contained in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Details of the type and method of sampling used are given on the attached logs.

INVESTIGATION METHODS

The following is a brief summary of investigation methods currently adopted by the Company and some comments on their use and application. All except test pits, hand auger drilling and portable dynamic cone penetrometers require the use of a mechanical drilling rig which is commonly mounted on a truck chassis.

Test Pits: These are normally excavated with a backhoe or a tracked excavator, allowing close examination of the insitu soils if it is safe to descend into the pit. The depth of penetration is limited to about 3m for a backhoe and up to 6m for an excavator. Limitations of test pits are the problems associated with disturbance and difficulty of reinstatement and the consequent effects on close-by structures. Care must be taken if construction is to be carried out near test pit locations to either properly recompact the backfill during construction or to design and construct the structure so as not to be adversely affected by poorly compacted backfill at the test pit location.

Hand Auger Drilling: A borehole of 50mm to 100mm diameter is advanced by manually operated equipment. Premature refusal of the hand augers can occur on a variety of materials such as hard clay, gravel or ironstone, and does not necessarily indicate rock level.

Continuous Spiral Flight Augers: The borehole is advanced using 75mm to 115mm diameter continuous spiral flight augers, which are withdrawn at intervals to allow sampling and insitu testing. This is a relatively economical means of drilling in clays and in sands above the water table. Samples are returned to the surface by the flights or may be collected after withdrawal of the auger flights, but they can be very disturbed and layers may become mixed. Information from the auger sampling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively lower reliability due to mixing or softening of samples by groundwater, or uncertainties as to the original depth of the samples. Augering below the groundwater table is of even lesser reliability than augering above the water table. Use can be made of a Tungsten Carbide (TC) bit for auger drilling into rock to indicate rock quality and continuity by variation in drilling resistance and from examination of recovered rock fragments.

Wash Boring: The borehole is usually advanced by a rotary bit, with water being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from "feel" and rate of penetration.

Mud Stabilised Drilling: Either Wash Boring or Continuous Core Drilling can use drilling mud as a circulating fluid to stabilise the borehole. The term "mud" encompasses a range of products ranging from bentonite to polymers such as Revert or Biogel. The mud tends to mask the cuttings and reliable identification is only possible from intermittent intact sampling (eg from SPT and U50 samples) or from rock coring, etc.

Continuous Core Drilling: A continuous core sample is obtained using a diamond tipped core barrel. Provided full core recovery is achieved (which is not always possible in very low strength rocks and granular soils), this technique provides a very reliable (but relatively expensive) method of investigation. In rocks, an NMLC triple tube core barrel, which gives a core of about 50mm diameter, is usually used with water flush. The length of core recovered is compared to the length drilled and any length not recovered is shown as CORE LOSS. The location of losses are determined on site by the supervising engineer; where the location is uncertain, the loss is placed at the top end of the drill run.

Standard Penetration Tests: Standard Penetration Tests (SPT) are used mainly in non-cohesive soils, but can also be used in cohesive soils as a means of indicating density or strength and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, "Methods of Testing Soils for Engineering Purposes" – Test F3.1.

The test is carried out in a borehole by driving a 50mm diameter split sample tube with a tapered shoe, under the impact of a 63kg hammer with a free fall of 760mm. It is normal for the tube to be driven in three successive 150mm increments and the 'N' value is taken as the number of blows for the last 300mm. In dense sands, very hard clays or weak rock, the full 450mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form:

- In the case where full penetration is obtained with successive blow counts for each 150mm of, say, 4, 6 and 7 blows, as

  N = 13
  4, 6, 7

- In a case where the test is discontinued short of full penetration, say after 15 blows for the first 150mm and 30 blows for the next 40mm, as

  N>30
  15, 30/40mm

The results of the test can be related empirically to the engineering properties of the soil.

Occasionally, the drop hammer is used to drive 50mm diameter thin walled sample tubes (U50) in clays. In such circumstances, the test results are shown on the borehole logs in brackets.

A modification to the SPT test is where the same driving system is used with a solid 60° tipped steel cone of the same diameter as the SPT hollow sampler. The solid cone can be continuously driven for some distance in soft clays or loose sands, or may be used where damage would otherwise occur to the SPT. The results of this Solid Cone Penetration Test (SCPT) are shown as "N_c" on the borehole logs, together with the number of blows per 150mm penetration.

Static Cone Penetrometer Testing and Interpretation: Cone penetrometer testing (sometimes referred to as a Dutch Cone) described in this report has been carried out using an Electronic Friction Cone Penetrometer (EFCP). The test is described in Australian Standard 1289, Test F5.1.

In the tests, a 35mm diameter rod with a conical tip is pushed continuously into the soil, the reaction being provided by a specially designed truck or rig which is fitted with an hydraulic ram system. Measurements are made of the end bearing resistance on the cone and the frictional resistance on a separate 134mm long sleeve, immediately behind the cone. Transducers in the tip of the assembly are electrically connected by wires passing through the centre of the push rods to an amplifier and recorder unit mounted on the control truck.

As penetration occurs (at a rate of approximately 20mm per second) the information is output as incremental digital records every 10mm. The results given in this report have been plotted from the digital data.

The information provided on the charts comprise:

- Cone resistance – the actual end bearing force divided by the cross sectional area of the cone – expressed in MPa.
- Sleeve friction – the frictional force on the sleeve divided by the surface area – expressed in kPa.
- Friction ratio – the ratio of sleeve friction to cone resistance, expressed as a percentage.

The ratios of the sleeve resistance to cone resistance will vary with the type of soil encountered, with higher relative friction in clays than in sands. Friction ratios of 1% to 2% are commonly encountered in sands and occasionally very soft clays, rising to 4% to 10% in stiff clays and peats. Soil descriptions based on cone resistance and friction ratios are only inferred and must not be considered as exact.

Correlations between EFCP and SPT values can be developed for both sands and clays but may be site specific.

Interpretation of EFCP values can be made to empirically derive modulus or compressibility values to allow calculation of foundation settlements.

Stratification can be inferred from the cone and friction traces and from experience and information from nearby boreholes etc. Where shown, this information is presented for general guidance, but must be regarded as interpretive. The test method provides a continuous profile of engineering properties but, where precise information on soil classification is required, direct drilling and sampling may be preferable.

Portable Dynamic Cone Penetrometers: Portable Dynamic Cone Penetrometer (DCP) tests are carried out by driving a rod into the ground with a sliding hammer and counting the blows for successive 100mm increments of penetration.

Two relatively similar tests are used:

- Cone penetrometer (commonly known as the Scala Penetrometer) – a 16mm rod with a 20mm diameter cone end is driven with a 9kg hammer dropping 510mm (AS1289, Test F3.2). The test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various Road Authorities.
- Perth sand penetrometer – a 16mm diameter flat ended rod is driven with a 9kg hammer, dropping 600mm (AS1289, Test F3.3). This test was developed for testing the density of sands (originating in Perth) and is mainly used in granular soils and filling.

LOGS

The borehole or test pit logs presented herein are an engineering and/or geological interpretation of the sub-surface conditions, and their reliability will depend to some extent on the frequency of sampling and the method of drilling or excavation. Ideally, continuous undisturbed sampling or core drilling will enable the most reliable assessment, but is not always practicable or possible to justify on economic grounds. In any case, the boreholes or test pits represent only a very small sample of the total subsurface conditions.

The attached explanatory notes define the terms and symbols used in preparation of the logs.

Interpretation of the information shown on the logs, and its application to design and construction, should therefore take into account the spacing of boreholes or test pits, the method of drilling or excavation, the frequency of sampling and testing and the possibility of other than "straight line" variations between the boreholes or test pits. Subsurface conditions between boreholes or test pits may vary significantly from conditions encountered at the borehole or test pit locations.

GROUNDWATER

Where groundwater levels are measured in boreholes, there are several potential problems:

- Although groundwater may be present, in low permeability soils it may enter the hole slowly or perhaps not at all during the time it is left open.
- A localised perched water table may lead to an erroneous indication of the true water table.
- Water table levels will vary from time to time with seasons or recent weather changes and may not be the same at the time of construction.
- The use of water or mud as a drilling fluid will mask any groundwater inflow. Water has to be blown out of the hole and drilling mud must be washed out of the hole or "reverted" chemically if water observations are to be made.

More reliable measurements can be made by installing standpipes which are read after stabilising at intervals ranging from several days to perhaps weeks for low permeability soils. Piezometers, sealed in a particular stratum, may be advisable in low permeability soils or where there may be interference from perched water tables or surface water.

FILL

The presence of fill materials can often be determined only by the inclusion of foreign objects (eg bricks, steel etc) or by distinctly unusual colour, texture or fabric. Identification of the extent of fill materials will also depend on investigation methods and frequency. Where natural soils similar to those at the site are used for fill, it may be difficult with limited testing and sampling to reliably determine the extent of the fill.

The presence of fill materials is usually regarded with caution as the possible variation in density, strength and material type is much greater than with natural soil deposits. Consequently, there is an increased risk of adverse engineering characteristics or behaviour. If the volume and quality of fill is of importance to a project, then frequent test pit excavations are preferable to boreholes.

LABORATORY TESTING

Laboratory testing is normally carried out in accordance with Australian Standard 1289 "Methods of Testing Soil for Engineering Purposes". Details of the test procedure used are given on the individual report forms.

ENGINEERING REPORTS

Engineering reports are prepared by qualified personnel and are based on the information obtained and on current engineering standards of interpretation and analysis. Where the report has been prepared for a specific design proposal (eg a three storey building) the information and interpretation may not be relevant if the design proposal is changed (eg to a twenty storey building). If this happens, the company will be pleased to review the report and the sufficiency of the investigation work.

Every care is taken with the report as it relates to interpretation of subsurface conditions, discussion of geotechnical aspects and recommendations or suggestions for design and construction. However, the Company cannot always anticipate or assume responsibility for:

- Unexpected variations in ground conditions – the potential for this will be partially dependent on borehole spacing and sampling frequency as well as investigation technique.
- Changes in policy or interpretation of policy by statutory authorities.
- The actions of persons or contractors responding to commercial pressures.

If these occur, the company will be pleased to assist with investigation or advice to resolve any problems occurring.

SITE ANOMALIES

In the event that conditions encountered on site during construction appear to vary from those which were expected from the information contained in the report, the company requests that it immediately be notified. Most problems are much more readily resolved when conditions are exposed that at some later stage, well after the event.

REPRODUCTION OF INFORMATION FOR CONTRACTUAL PURPOSES

Attention is drawn to the document "Guidelines for the Provision of Geotechnical Information in Tender Documents", published by the Institution of Engineers, Australia. Where information obtained from this investigation is provided for tendering purposes, it is recommended that all information, including the written report and discussion, be made available. In circumstances where the discussion or comments section is not relevant to the contractual situation, it may be appropriate to prepare a specially edited document. The company would be pleased to assist in this regard and/or to make additional report copies available for contract purposes at a nominal charge.

Copyright in all documents (such as drawings, borehole or test pit logs, reports and specifications) provided by the Company shall remain the property of Jeffery and Katauskas Pty Ltd. Subject to the payment of all fees due, the Client alone shall have a licence to use the documents provided for the sole purpose of completing the project to which they relate. License to use the documents may be revoked without notice if the Client is in breach of any objection to make a payment to us.

REVIEW OF DESIGN

Where major civil or structural developments are proposed <u>or</u> where only a limited investigation has been completed <u>or</u> where the geotechnical conditions/ constraints are quite complex, it is prudent to have a joint design review which involves a senior geotechnical engineer.

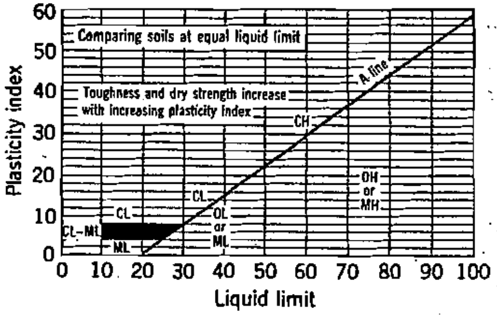
SITE INSPECTION

The company will always be pleased to provide engineering inspection services for geotechnical aspects of work to which this report is related.

Requirements could range from:

i) a site visit to confirm that conditions exposed are no worse than those interpreted, to
ii) a visit to assist the contractor or other site personnel in identifying various soil/rock types such as appropriate footing or pier founding depths, or
iii) full time engineering presence on site.

UNIFIED SOIL CLASSIFICATION TABLE

Field Identification Procedures (Excluding particles larger than 75 µm and basing fractions on estimated weights)							Group Symbols[a]	Typical Names	Information Required for Describing Soils	Laboratory Classification Criteria			
(The 75 µm sieve size is about the smallest particle visible to naked eye)	Coarse-grained soils More than half of material is *larger* than 75 µm sieve size[b]	Gravels More than half of coarse fraction is larger than 4 mm sieve size	Clean gravels (little or no fines)	Wide range in grain size and substantial amounts of all intermediate particle sizes			*GW*	Well graded gravels, gravel-sand mixtures, little or no fines	Give typical name: indicate approximate percentages of sand and gravel; maximum size; angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbols in parentheses For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics Example: *Silty sand*, gravelly; about 20% hard, angular gravel particles 12 mm maximum size; rounded and subangular sand grains coarse to fine, about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (*SM*)	Use grain size curve in identifying the fractions as given under field identification	Determine percentages of gravel and sand from grain size curve Depending on percentage of fines (fraction smaller than 75 µm sieve size) coarse grained soils are classified as follows: Less than 5% *GW, GP, SW, SP* More than 12% *GM, GC, SM, SC* 5% to 12% *Borderline* cases requiring use of dual symbols	$C_U = \frac{D_{60}}{D_{10}}$ Greater than 4 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
				Predominantly one size or a range of sizes with some intermediate sizes missing			*GP*	Poorly graded gravels, gravel-sand mixtures, little or no fines				Not meeting all gradation requirements for *GW*	
			Gravels with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures see *ML* below)			*GM*	Silty gravels, poorly graded gravel-sand-silt mixtures				Atterberg limits below "A" line, or *PI* less than 4	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
				Plastic fines (for identification procedures, see *CL* below)			*GC*	Clayey gravels, poorly graded gravel-sand-clay mixtures				Atterberg limits above "A" line, with *PI* greater than 7	
		Sands More than half of coarse fraction is smaller than 4 mm sieve size	Clean sands (little or no fines)	Wide range in grain sizes and substantial amounts of all intermediate particle sizes			*SW*	Well graded sands, gravelly sands, little or no fines				$C_U = \frac{D_{60}}{D_{10}}$ Greater than 6 $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
				Predominantly one size or a range of sizes with some intermediate sizes missing			*SP*	Poorly graded sands, gravelly sands, little or no fines				Not meeting all gradation requirements for *SW*	
			Sands with fines (appreciable amount of fines)	Nonplastic fines (for identification procedures, see *ML* below)			*SM*	Silty sands, poorly graded sand-silt mixtures				Atterberg limits below "A" line or *PI* less than 5	Above "A" line with *PI* between 4 and 7 are *borderline* cases requiring use of dual symbols
				Plastic fines (for identification procedures, see *CL* below)			*SC*	Clayey sands, poorly graded sand-clay mixtures				Atterberg limits below "A" line with *PI* greater than 7	
		Identification Procedures on Fraction Smaller than 380 µm Sieve Size											
	Fine-grained soils More than half of material is *smaller* than 75 µm sieve size			Dry Strength (crushing characteristics)	Dilatancy (reaction to shaking)	Toughness (consistency near plastic limit)							
		Silts and clays liquid limit less than 50		None to slight	Quick to slow	None	*ML*	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity	Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; colour in wet condition, odour if any, local or geologic name, and other pertinent descriptive information, and symbol in parentheses For undisturbed soils add information on structure, stratification, consistency in undisturbed and remoulded states, moisture and drainage conditions Example: *Clayey silt*, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (*ML*)		 Plasticity chart for laboratory classification of fine grained soils		
				Medium to high	None to very slow	Medium	*CL*	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays					
				Slight to medium	Slow	Slight	*OL*	Organic silts and organic silt-clays of low plasticity					
		Silts and clays liquid limit greater than 50		Slight to medium	Slow to none	Slight to medium	*MH*	Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts					
				High to very high	None	High	*CH*	Inorganic clays of high plasticity, fat clays					
				Medium to high	None to very slow	Slight to medium	*OH*	Organic clays of medium to high plasticity					
	Highly Organic Soils			Readily identified by colour, odour, spongy feel and frequently by fibrous texture			*Pt*	Peat and other highly organic soils					

NOTE: 1) Soils possessing characteristics of two groups are designated by combinations of group symbols (e.g. GW-GC, well graded gravel-sand mixture with clay fines).

2) Soils with liquid limits of the order of 35 to 50 may be visually classified as being of medium plasticity.

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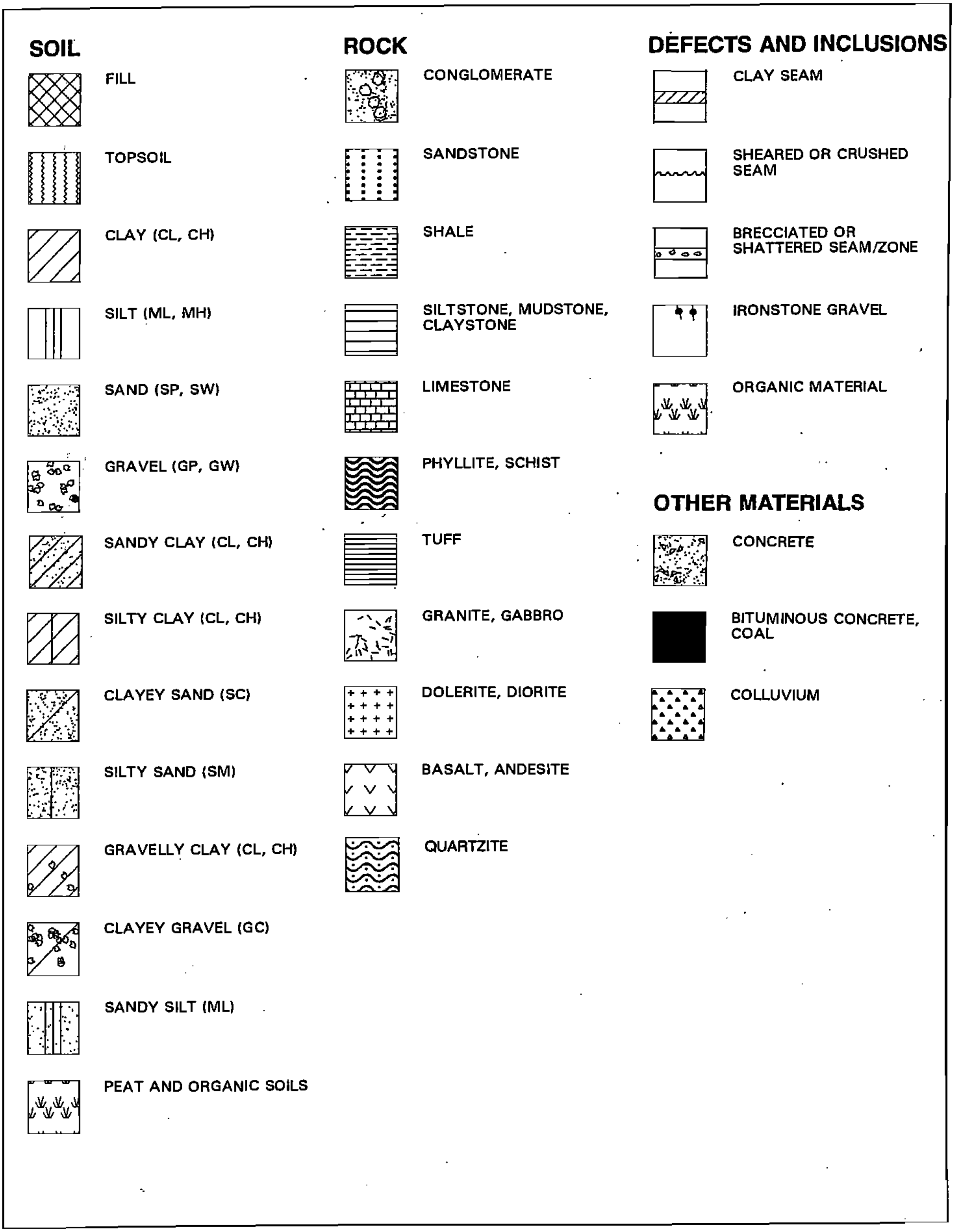
GRAPHIC LOG SYMBOLS FOR SOILS AND ROCKS



SOIL

- FILL
- TOPSOIL
- CLAY (CL, CH)
- SILT (ML, MH)
- SAND (SP, SW)
- GRAVEL (GP, GW)
- SANDY CLAY (CL, CH)
- SILTY CLAY (CL, CH)
- CLAYEY SAND (SC)
- SILTY SAND (SM)
- GRAVELLY CLAY (CL, CH)
- CLAYEY GRAVEL (GC)
- SANDY SILT (ML)
- PEAT AND ORGANIC SOILS

ROCK

- CONGLOMERATE
- SANDSTONE
- SHALE
- SILTSTONE, MUDSTONE, CLAYSTONE
- LIMESTONE
- PHYLLITE, SCHIST
- TUFF
- GRANITE, GABBRO
- DOLERITE, DIORITE
- BASALT, ANDESITE
- QUARTZITE

DEFECTS AND INCLUSIONS

- CLAY SEAM
- SHEARED OR CRUSHED SEAM
- BRECCIATED OR SHATTERED SEAM/ZONE
- IRONSTONE GRAVEL
- ORGANIC MATERIAL

OTHER MATERIALS

- CONCRETE
- BITUMINOUS CONCRETE, COAL
- COLLUVIUM

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LOG SYMBOLS

LOG COLUMN	SYMBOL	DEFINITION
Groundwater Record	▼	Standing water level. Time delay following completion of drilling may be shown.
	—C—	Extent of borehole collapse shortly after drilling.
	▶—	Groundwater seepage into borehole or excavation noted during drilling or excavation.
Samples	ES	Soil sample taken over depth indicated, for environmental analysis.
	U50	Undisturbed 50mm diameter tube sample taken over depth indicated.
	DB	Bulk disturbed sample taken over depth indicated.
	DS	Small disturbed bag sample taken over depth indicated.
Field Tests	N = 17 4, 7, 10	Standard Penetration Test (SPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration. 'R' as noted below.
	N_c = 5 7 3R	Solid Cone Penetration Test (SCPT) performed between depths indicated by lines. Individual figures show blows per 150mm penetration for 60 degree solid cone driven by SPT hammer. 'R' refers to apparent hammer refusal within the corresponding 150mm depth increment.
	VNS = 25	Vane shear reading in kPa of Undrained Shear Strength.
	PID = 100	Photoionisation detector reading in ppm (Soil sample headspace test).
Moisture Condition (Cohesive Soils)	MC>PL	Moisture content estimated to be greater than plastic limit.
	MC≈PL	Moisture content estimated to be approximately equal to plastic limit.
	MC<PL	Moisture content estimated to be less than plastic limit.
(Cohesionless Soils)	D	DRY - runs freely through fingers.
	M	MOIST - does not run freely but no free water visible on soil surface.
	W	WET - free water visible on soil surface.
Strength (Consistency) Cohesive Soils	VS	VERY SOFT - Unconfined compressive strength less than 25kPa
	S	SOFT - Unconfined compressive strength 25-50kPa
	F	FIRM - Unconfined compressive strength 50-100kPa
	St	STIFF - Unconfined compressive strength 100-200kPa
	VSt	VERY STIFF - Unconfined compressive strength 200-400kPa
	H	HARD - Unconfined compressive strength greater than 400kPa
	( )	Bracketed symbol indicates estimated consistency based on tactile examination or other tests.
Density Index/ Relative Density (Cohesionless Soils)		**Density Index (I_D) Range (%)** **SPT 'N' Value Range (Blows/300mm)**
	VL	Very Loose <15 0-4
	L	Loose 15-35 4-10
	MD	Medium Dense 35-65 10-30
	D	Dense 65-85 30-50
	VD	Very Dense >85 >50
	( )	Bracketed symbol indicates estimated density based on ease of drilling or other tests.
Hand Penetrometer Readings	300 250	Numbers indicate individual test results in kPa on representative undisturbed material unless noted otherwise.
Remarks	'V' bit	Hardened steel 'V' shaped bit.
	'TC' bit	Tungsten carbide wing bit.
	┬ 60	Penetration of auger string in mm under static load of rig applied by drill head hydraulics without rotation of augers.

Ref: Standard Sheets Log Symbols
August 2001

Jeffery and Katauskas Pty Ltd

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LOG SYMBOLS

ROCK MATERIAL WEATHERING CLASSIFICATION

TERM	SYMBOL	DEFINITION
Residual Soil	RS	Soil developed on extremely weathered rock; the mass structure and substance fabric are no longer evident; there is a large change in volume but the soil has not been significantly transported.
Extremely weathered rock	XW	Rock is weathered to such an extent that it has "soil" properties, ie it either disintegrates or can be remoulded, in water.
Distinctly weathered rock	DW	Rock strength usually changed by weathering. The rock may be highly discoloured, usually by ironstaining. Porosity may be increased by leaching, or may be decreased due to deposition of weathering products in pores.
Slightly weathered rock	SW	Rock is slightly discoloured but shows little or no change of strength from fresh rock.
Fresh rock	FR	Rock shows no sign of decomposition or staining.

ROCK STRENGTH

Rock strength is defined by the Point Load Strength Index (Is 50) and refers to the strength of the rock substance in the direction normal to the bedding. The test procedure is described by the International Journal of Rock Mechanics, Mining, Science end Geomechanics. Abstract Volume 22, No 2, 1985.

TERM	SYMBOL	Is (50) MPa	FIELD GUIDE
Extremely Low:	EL		Easily remoulded by hand to a material with soil properties.
		0.03	
Very Low:	VL		May be crumbled in the hand. Sandstone is "sugary" and friable.
		0.1	
Low:	L		A piece of core 150mm long x 50mm dia. may be broken by hand and easily scored with a knife. Sharp edges of core may be friable and break during handling.
		0.3	
Medium Strength:	M		A piece of core 150mm long x 50mm dia. can be broken by hand with difficulty. Readily scored with knife.
		1	
High:	H		A piece of core 150mm long x 50mm dia. core cannot be broken by hand, can be slightly scratched or scored with knife; rock rings under hammer.
		3	
Very High:	VH		A piece of core 150mm long x 50mm dia. may be broken with hand-held pick after more than one blow. Cannot be scratched with pen knife; rock rings under hammer.
		10	
Extremely High:	EH		A piece of core 150mm long x 50mm dia. is very difficult to break with hand-held hammer. Rings when struck with a hammer.

ABBREVIATIONS USED IN DEFECT DESCRIPTION

ABBREVIATION	DESCRIPTION	NOTES
Be	Bedding Plane Parting	Defect orientations measured relative to the normal to the long core axis (ie relative to horizontal for vertical holes)
CS	Clay Seam	
J	Joint	
P	Planar	
Un	Undulating	
S	Smooth	
R	Rough	
IS	Ironstained	
XWS	Extremely Weathered Seam	
Cr	Crushed Seam	
60t	Thickness of defect in millimetres	

Ref: Standard Sheets Log Symbols
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